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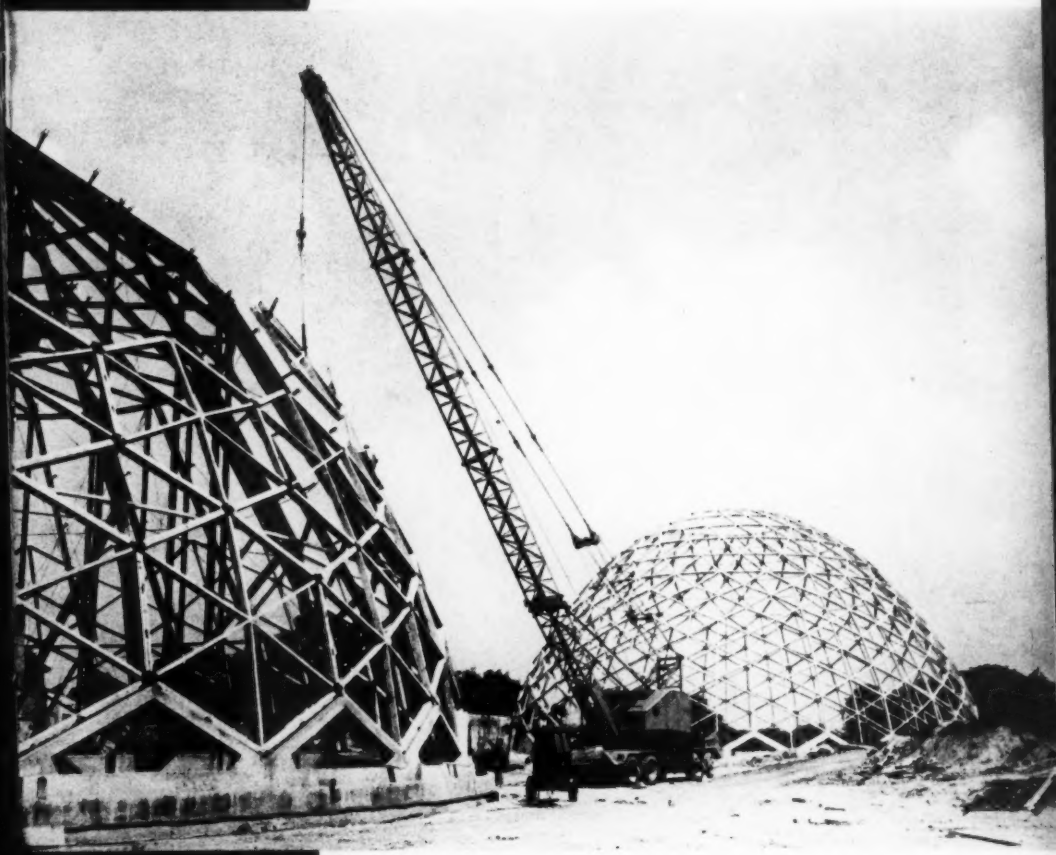

Report of the committee
on nominations appears
on p. 3 of the News Letter

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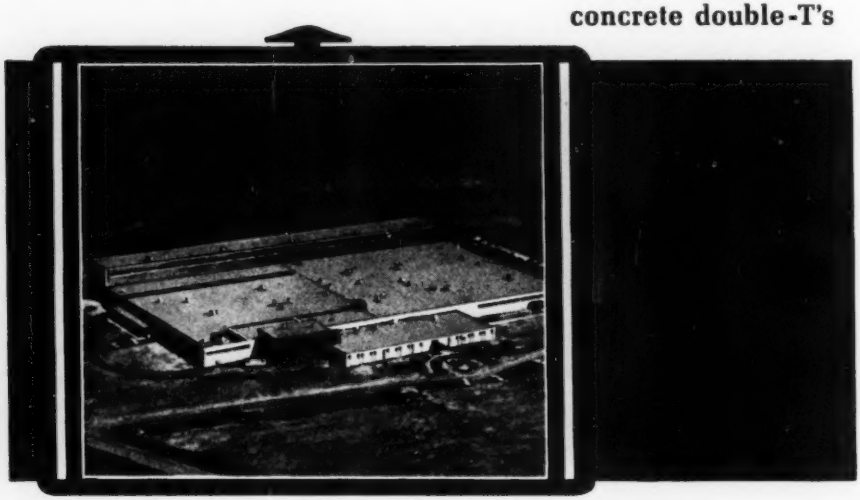

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On the Cover

Erection of hexagonal piece on one of three domes at Milwaukee's Botanical Garden Development at Mitchell Park. A detailed description of the design, casting, and erection of this project is given in the article beginning on p. 533.

Coming next month in the JOURNAL

J. NEILS THOMPSON, HUDSON MATLOCK, and A. ANTHONY TOPRAC open the December issue with a structural research paper on the "Welding of Reinforcing Steel Between Precast Concrete Units."

●

"Fifty Year Compression Tests of Concrete," by M. O. WITHEY reports on a long-time study conducted over the past 50 years to determine the effect of age and certain curing conditions on the compressive strength of concrete made with portland cement.

●

J. K. SANT and RICHARD W. BLETZACKER review results of "Experimental Study of the Lateral Stability of Reinforced Concrete Beams."

●

R. S. SANDHU describes the principles in the "Design of Concrete Linings for Large Underground Conduits," successfully employed in the design of the tunnel system at Bhakra Dam, India.

●

DONALD F. GRIFFIN and ROBERT L. HENRY discuss the "Integral Sodium Chloride Effect on Strength, Water Vapor Transmission, and Efflorescence of Concrete."

●

"Steady State Thermal Stresses in Rigid Frames," by JOSEPH J. GENNARO presents a method for finding the moments and stresses developed in beams and rigid frames subject to linear variations of temperature between the lower and upper edges of the beam.

●

Discussion of papers published in the April, May, and June, 1961 issues will appear in the Part 2, December JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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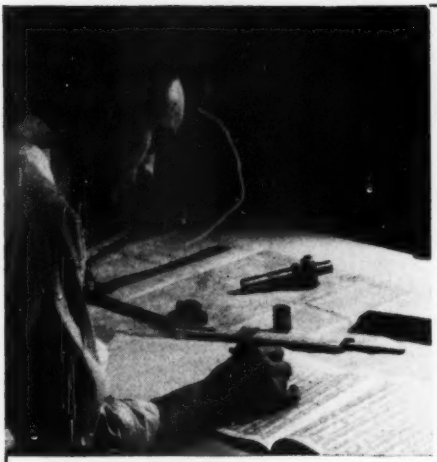

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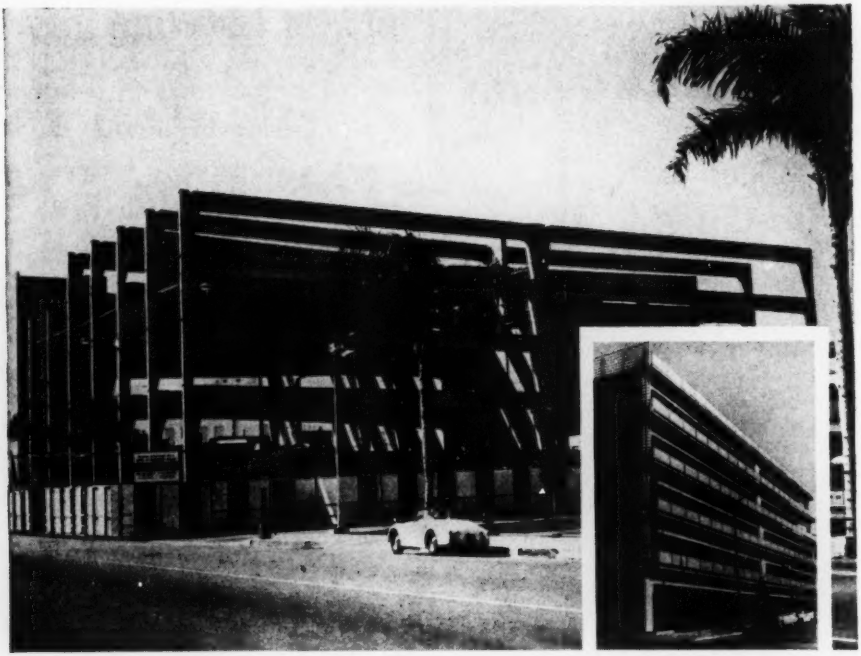

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Selection and Use of Aggregates for Concrete

Reported by ACI Committee 621

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Presents available information on aggregates in four categories. (1) Evaluation of aggregate properties in terms of their influence on the properties of concrete. (2) Methods of determining aggregate properties and the limitations of these methods. (3) Features of aggregate preparation and handling which have a bearing on concrete quality and uniformity. (4) Selection of aggregate.

The report is limited to sand, gravel, crushed stone, and air-cooled blast-furnace slag aggregate. Lightweight aggregate and special heavy aggregate are not covered.

■ NOTWITHSTANDING THAT MINERAL aggregates represent the major volume of concrete—approximately 75 percent—the important role they serve as the principal ingredient is often overlooked because their cost is so much less than that of cement. Accordingly, Committee 621, Aggregates, was asked to prepare a report which will assist in the selection and use of aggregates for concrete.

It is the purpose of the committee that this report shall be a statement of information on aggregate that will assist in evaluating properties of aggregate in terms of their influence on properties of concrete, discuss features of preparation and handling which have a bearing on concrete quality and uniformity, and, in general, summarize what should concern the user about aggregate when he sets forth to do a first-class concrete job. The scope of the report is limited to sand, gravel,

crushed stone, and air-cooled blast-furnace slag.* It is not intended to cover the use of lightweight aggregate or the special heavy aggregates.

This discussion is divided into four parts: (I) properties of concrete influenced by aggregate properties, (II) methods of determining aggregate properties and their limitations, (III) features of preparation and handling which have a bearing on concrete quality and uniformity, and (IV) selection of aggregate. Although the user does not specify the methods and equipment used in aggregate preparation or beneficiation, his requirements for aggregate possessing certain characteristics or properties will, in many cases, influence the processing of clean, sound aggregate of uniform quality. Part III is not intended as a guide for aggregate producers. It is included for the benefit of the user who frequently must do some handling of the aggregate.†

To the extent practicable, the selection of an aggregate should be based on knowledge of its significant properties as determined by service record, laboratory tests, and petrographic examination. Service record, when available in sufficient detail, is a most valuable aid in guiding judgment. To be significant, the record should cover structures with concrete proportions and exposures similar to those anticipated for the proposed work. Petrographic or other suitable procedures should be used to determine whether or not the aggregate in the structure and that proposed for use are sufficiently similar to make the service record significant. Further, it should not be overlooked that unfavorable service of old concrete without entrained air may not accurately indicate performance of concrete with the benefits of proper air entrainment.

A summary of the available information on mineral aggregates was published in 1948[1] and a summary of data on aggregate properties and their influence on the behavior of concrete is contained in "Significance of Tests and Properties of Concrete and Concrete Aggregates."[2] It contains nine papers on tests and properties of aggregates.

PART I — PROPERTIES OF CONCRETE INFLUENCED BY AGGREGATE PROPERTIES

Since aggregates occupy three-quarters of the volume of concrete, it is to be expected that properties of the aggregate have a major effect on the properties of concrete. Eight properties of concrete are discussed below, and the pertinent aggregate properties are considered.

Durability

For many exposures the most important property of concrete is its durability. There are several aspects to the problem and practically all of these are influenced by properties of the aggregate.

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*Definitions of blast-furnace slag, coarse aggregate, crushed gravel, crushed stone, fine aggregate, gravel, and sand are given in "Standard Definitions of Terms Relating to Concrete and Concrete Aggregates," ASTM C 125-58. Air-cooled slag is the product that results when molten slag is deposited in pits or banks for solidification under atmospheric conditions. It may then be further cooled by the application of water.

†A description of handling procedures is contained in (ACI 614-59), Reference 78.

ACI Committee 621, Aggregates, was first organized in 1936 under the title "Aggregate, Selection, Preparation, Handling, and Use." The present committee was reorganized in 1955 under its present title. At that time the committee was given the mission of preparing a statement of information on aggregate that will evaluate properties of aggregate in terms of their influence on properties of concrete, discuss features of preparation and handling which had a bearing on concrete quality and uniformity, and in general, summarize what should concern the user about aggregate when he sets forth to do a first-class job.

Previous work of the committee yielded papers by committee members on production of sand and gravel, crushed stone, and slag.

1. Resistance to freezing and thawing — Concrete containing good aggregate will not be resistant to freezing and thawing if the paste is inadequate; nor will concrete containing a frost-resistant paste if it contains unsound aggregate particles which are critically saturated. A particle is considered to be critically saturated when there is insufficient unfilled pore space to accommodate the expansion of water which accompanies freezing. The property of "soundness," usually defined as the ability of an aggregate to resist large or permanent changes of volume when subjected to freezing and thawing, heating and cooling, or wetting and drying, is related to the porosity, absorption, and pore structure of the aggregate. Rocks that can absorb water so as to become critically saturated are potentially vulnerable to freezing. Current laboratory studies and theoretical analysis suggest that for any rate of freezing there is a critical particle size, dependent on the permeability and tensile strength, above which the particle will fail if critically saturated.[3] For fine-grained materials with low permeability the critical size may be in the range of normal aggregate sizes. For coarse-grained materials or materials with the capillary system interrupted by numerous macropores, the critical size might be so large so as to be of no consequence even though the absorption might be high. If potentially vulnerable aggregates are dry when used and are subjected to periodic drying in service, they may never become critically saturated. In this connection the paramount importance of an air-entrained paste in concrete exposed to frost action should be fully recognized.

When the aggregate contains only a few particles of unfavorable pore characteristics, freezing frequently produces, instead of general disintegration, the phenomenon known as "pop-outs" in which coarse aggregate particles near the surface push off the surface layer of mortar when they expand, leaving holes in the surface. Chert particles of low specific gravity, limestone containing clay, and shaly materials are well known for this behavior.

2. Resistance to wetting and drying — The influence of aggregate on durability of concrete subjected to wetting and drying is also controlled by pore structure of the aggregate. Although this problem is not nearly as serious as the problem of freezing and thawing, the differential

swelling accompanying moisture gain of a material with a large amount of capillary absorption may be sufficient to cause failure of the surrounding paste. The amount of stress developed is proportional to the modulus of elasticity of the aggregate. In some cases pop-outs may occur.

3. Resistance to heating and cooling — Heating and cooling induce stresses in any material. If the temperature range is great enough, damage will result. For aggregates commonly used and for temperature changes ordinarily encountered, this is not a critical factor in concrete. However, in addition to this over-all effect, there has been theoretical speculation, and some laboratory evidence and field observations have been interpreted to support it,[4-8] that large differences between the values for paste and aggregate of the thermal coefficient of expansion or thermal diffusivity should produce significant stresses in concrete when subjected to normal temperature change. There is no general agreement that a problem exists here. In interpreting laboratory tests and field observations, it is difficult to isolate thermal effects from others such as moisture changes. Although the usual practice is not to restrict the expansion coefficient of aggregate for normal temperature exposure, aggregates with coefficients which are extremely high or low may require investigation before use in certain types of structures.

4. Abrasion resistance — Abrasion resistance is another property for which a high-quality well-cured paste is paramount. However, the hardness of the aggregate is an important factor and abrasion resistant floors and other concrete surfaces cannot be expected when soft aggregates are used.

5. Alkali-aggregate reaction expansion — Deterioration resulting from the alkali-aggregate reaction is recognized as a serious problem in certain geographic areas, principally in the southern and western parts of the United States. It may occur elsewhere in areas where the effect is concealed by the results of frost action. Reaction between alkalies in the cement, and certain siliceous constituents in some aggregates, produces harmful expansion. On the other hand, evidence of reaction has been observed in concretes in which no damage has occurred. The problem can be prevented by the use of a low-alkali cement or by the addition to the mixture of an adequate amount of a suitable pozzolan. The user confronted with the problem is referred to the extensive literature on the subject.[9]

6. Fire resistance — Concretes containing the aggregates within the scope of this report show little difference in fire resistance when exposed in a dry condition. The extensive field observations and laboratory tests[10-32] indicate a slight advantage for blast furnace slag, calcareous aggregates, and aggregates of igneous origin over siliceous aggregates of sedimentary or metamorphic origin.

7. *Acid resistance* — Acid resistance is more intimately associated with the cement paste than with the aggregates. However acid-resistant aggregates are required for special uses.[33]

Strength

Perhaps the second most important property of concrete, and the one for which values are most frequently specified, is strength. The types of strengths usually considered are compressive and flexural. Strength depends largely on the strength of the cement paste, and on the bond between the paste and aggregate. The strength of the aggregate also affects the strength of the concrete, but for many aggregates the differences are relatively small as compared to those resulting from differences in strength of the cement pastes in which they are used. Consideration of factors affecting the strength of the paste is beyond the scope of this report. The bond between the paste and aggregate tends to set an upper limit on the strength of concrete that can be obtained with a given set of materials, particularly in the case of flexural strength. Bond is influenced by the surface texture and cleanness of the aggregate. A rough-textured surface normally bonds better than a smooth surface. Coatings which adhere to the aggregate during mixing may interfere with bond. Many coatings have no deleterious effect. Those which are removed during mixing have the effect of augmenting the fines in the aggregates. Those which remain on the aggregate particle surface after mixing and placing have no particular effect unless they are of such a nature so as to interfere with bond or are of a chemical composition which will produce a deleterious reaction with alkalies in cement.[34] Clayey coatings will normally interfere with bond, while nonadherent dust coatings increase the water demand as a consequence of the increase in fines.[35]

While angular particles and those having rough, vesicular surfaces have a higher water requirement than rounded material, nevertheless crushed and uncrushed aggregates generally give substantially the same compressive strengths for a given cement factor. Some aggregates, which are otherwise suitable, have a higher than normal water requirement because of unfavorable grading characteristics or the presence of a large proportion of flat or elongated particles. With such materials it is necessary to use a higher than normal cement factor to avoid an excessively high water-cement ratio and as a result, insufficient strength. Water requirement may also be increased by nonadherent coatings and by poor abrasion resistance of the aggregate in that both increase the quantity of fines in the mixer.

There is experimental evidence[36] to show that at a fixed water-cement ratio strength decreases slightly as maximum size of aggregate increases

particularly for sizes larger than 1½ in. However, for the same cement content, this apparent advantage of the smaller size may not be shown because of the offsetting effects of the increased quantity of mixing water required.

Shrinkage

The amount of shrinkage occurring during drying of concrete is dependent on the shrinkage potential of both the cement paste and aggregate, the total volume of paste in the concrete, and possibly the modulus of elasticity of the aggregate. Theoretical studies indicate that the higher the modulus of elasticity, the greater should be the restraint offered to the shrinking paste and hence the less the shrinkage measured in the concrete,[37,38] although the magnitude of the effect has not been determined experimentally. Since the quantity of paste depends on water requirement of the aggregate, such properties as maximum size, particle shape, grading, and cleanness are related to shrinkage. In addition, there is evidence that expansive clays, if present in aggregate, increase shrinkage to a greater extent than can be accounted for by increased water requirement.

Thermal properties

The coefficient of thermal expansion, specific heat, thermal conductivity, and thermal diffusivity of concrete are largely dependent on these same properties of the aggregate.

It has been demonstrated that the coefficient of thermal expansion can be computed approximately as the average of the values for the constituents weighted in proportion to the volumes present.[39, 40] It has also been demonstrated that each of the materials composing the concrete contributes to the conductivity and specific heat of the product in proportion to the amount of the material present. Necessarily diffusivity is similarly affected.[41]

Unit weight

The unit weight of the concrete depends on the specific gravity of the aggregate, on the amount of air entrained, and on those properties discussed above which determine water requirement. Since the specific gravity of cement paste is less than that of normal aggregate, unit weight normally increases as the amount of paste decreases.

Modulus of elasticity

The modulus of elasticity of concrete is to some extent dependent on the modulus of elasticity and Poisson's ratio of the aggregate. However, for a given cement paste the modulus of elasticity of the aggregate has less effect on the modulus of elasticity of the concrete than can be accounted for by the volumetric proportions of aggregate in the concrete.[42]

Slipperiness

Slipperiness of pavements seems to be almost entirely related to the tendency of some aggregates to become polished as the concrete surface is worn by traffic.[43, 44]

Economy

Economy of concrete is influenced by the quantity of cement necessary to produce required strength or other properties, by the availability or proximity of suitable material, and by the amount of processing required to produce suitable aggregate. Although well-shaped aggregates whether angular or rounded, graded within the limits of generally accepted specifications, will produce concretes of comparable quality at a given cement factor, it is pointed out in the report of ACI Committee 613[45] that with aggregates having characteristics which produce abnormally high water requirements, it is necessary to increase the cement content to maintain a fixed water-cement ratio.

The information presented in this section is summarized in Table 1.

PART II — METHODS OF DETERMINING AGGREGATE PROPERTIES AND THEIR LIMITATIONS

The aggregate properties listed in Table 1 are subject to either direct or indirect measurement in the laboratory. Some methods are commonly employed for specification purposes; others are not used for this purpose either because they require specialized equipment and techniques or because there is no general agreement on proper specification limits for the properties measured. Most emphasis here will be put on the specification tests. The others, while important for research, are not available to most concrete users.

Properties frequently specified or tests frequently performed

1. Particle size distribution, "Grading," "Gradation," "Sieve Analysis." (ASTM C 137, AASHO T27, ASA A37.8) — The particle size distribution has a major effect on the water requirement of concrete made from a given aggregate and thereby an effect on all the properties of concrete related to water requirement. It also has an important effect on the workability and finishing characteristics of fresh concrete.[46] The sieve analysis is probably the most frequently made of all aggregate tests. The particle size distribution is determined directly by passing samples of the aggregate through a nest of sieves of successively smaller openings and weighing the material retained on each sieve. This procedure also establishes the maximum size of particles present in the sample.

2. Specific gravity — An accurate knowledge of the specific gravity of both fine and coarse aggregate is required to calculate the batch weights needed to provide the desired absolute volumes of materials. Bulk spe-

cific gravity (saturated-surface-dry basis) is normally used in connection with concrete aggregates.

a. Coarse Aggregate (ASTM C 127, AASHO T85, ASA A37.5). The specific gravity of saturated surface-dry coarse aggregate is determined by weighing the material in air and in water.

TABLE 1 — PROPERTIES OF CONCRETE INFLUENCED BY AGGREGATE PROPERTIES

Concrete property	Relevant aggregate property
Durability:	
Resistance to freezing and thawing	Soundness Porosity Pore structure Permeability Degree of saturation Tensile strength Texture and structure Presence of clay
Resistance to wetting and drying	Pore structure Modulus of elasticity
Resistance to heating and cooling	Coefficient of thermal expansion
Abrasion Resistance	Hardness
Alkali-aggregate reaction	Presence of particular siliceous constituents
Strength	Strength Surface texture Cleanness Particle shape Maximum size
Shrinkage	Modulus of elasticity Particle shape Grading Cleanness Maximum size Presence of clay
Coefficient of thermal expansion	Coefficient of thermal expansion Modulus of elasticity
Thermal conductivity	Thermal conductivity
Specific heat	Specific heat
Unit weight	Specific gravity Particle shape Grading Maximum size
Modulus of elasticity	Modulus of elasticity Poisson's ratio
Slipperiness	Tendency to polish
Economy	Particle shape Grading Maximum size Amount of processing required Availability

b. Fine aggregate (ASTM C 128, AASHO T85, ASA 37.6). The method used for coarse aggregate does not lend itself to fine aggregate because of difficulty of containing the material for the weight in water determination. The procedure used instead is to place a known weight of saturated surface-dry sand into a vessel of known volume and to determine the volume of the material by measuring the amount of water required to fill the vessel.

3. Unit weight (ASTM C 29, AASHO T19, ASA A37.16) — The dry rodded unit weight, together with the specific gravity, provides a means for computing the voids in a unit volume of aggregate. In mixture proportioning it provides a means for estimating the amount of mortar required for a given coarse aggregate. The test is performed by determining the weight of aggregate required to fill a calibrated measure.

4. Absorption (ASTM C 127, C 128; AASHO T84, T85, ASA A37.5, A37.6) — While aggregate absorption data are frequently used as an aid in assessing the probable durability of concrete exposed to freezing and thawing, such a procedure must be applied with caution. Although most rocks with low absorption are durable, many durable rocks also have a high absorption. The structure of the pores as well as their total volume is important. A knowledge of absorption is essential to field control when surface moisture is determined by drying. Absorption is determined directly from the weights of a sample in the saturated surface-dry and oven-dry conditions.

5. Surface moisture — A knowledge of surface moisture is essential to field control of mixing water. Water carried into the mixer on the aggregate must be subtracted from the weight of added water while the scale settings for aggregate must be increased an equal amount. Surface moisture is commonly determined in the field by oven drying a sample and subtracting from the total water content the absorption as determined above. Devices are available in which chemicals added to a sample in a closed container react with water to produce a pressure which is a function of the amount of water present. Electrical devices for measuring the moisture instantaneously while the aggregate is in the weighing hopper also exist; these are based on resistance, dielectric, or neutron absorption principles. While none has been universally accepted as providing correct absolute values under all conditions, they are helpful in indicating when changes in surface moisture occur.

There is a standard test for a direct measurement of surface moisture of fine aggregate, ASTM C 70, AASHO T142, ASA A37.31. The amount of water that must be added to a sample to completely fill a calibrated vessel is used as an indication of the amount of water originally present on a sample of known weight or solid volume. The method is not in wide use because of the difficulty in removing all the air from the sample as water is added.

6. Soundness

a. Sulfate soundness test (ASTM C 88, AASHO T104, ASA A37.23). This widely used test consists of alternately immersing an aggregate sample in a solution of sodium or magnesium sulfate and drying it in an oven. The enlargement of salt crystals in the aggregate by rehydration during immersion after oven drying is presumed to simulate the increase in volume of water on freezing in the aggregate pores or cracks. Poor performance is indicated if, after test, a large part of the coarse aggregate sample will pass a sieve with openings 5/6 of those on which it was originally retained and if after test a large part of the sand sample will pass sieves on which it was originally retained. The choice of sodium or magnesium sulfate is largely one of individual preference. They do not yield results which are directly comparable.

Although specifications frequently contain acceptance limits, the sulfate soundness test has not been conspicuously successful in evaluating the resistance of aggregate to freezing and thawing in concrete. Its failure, apparently, is due not to the lack of similarity between sulfate crystal growth and ice formation but to the fact that in the sulfate test the aggregate is tested in the unconfined state. Unconfined freezing tests of aggregate particles have been no more successful than the sulfate test. Aggregate in concrete is surrounded by the fine-grained cement paste of extremely low permeability which greatly alters the exposure conditions.

b. Freezing and thawing of concrete. (ASTM C 290, C 291, C 292; AASHO T161). Freezing and thawing tests of aggregate in concrete probably provide the best measure of the soundness of aggregates, although no test has yet been standardized for general use. These methods permit the comparison of aggregates by subjecting air-entrained concrete containing samples of the aggregates to alternate cycles of freezing and thawing. Deterioration, if any, is measured by the progressive reduction in the dynamic modulus of elasticity. It may also be determined by periodic measurements of weight loss or length. The methods all require the use of moist-cured specimens, and they specify thawing in water. They differ in the speed of the cycle and in whether the freezing is done in water or air. The choice of a method is largely a matter of personal preference and available equipment. The use of accelerated freezing and thawing tests has been limited largely to within-laboratory comparison of aggregates. An evaluation of within-laboratory, between-laboratory and between-method reproducibility has been published by the Highway Research Board;[47] other data are given by Trudsø.[48]

Objections have been raised to certain aspects of these tests, and an alternate test[3] has been proposed and is in limited use. The chief

objection to the usual tests is in the use of initially saturated specimens whereas most concrete in service is partially dried at the start of the winter. The alternate test requires that the specimens be in the same moisture condition as the prototype at the start of test. They are then soaked in water continuously and are periodically subjected to a single cycle of freezing to determine whether critical saturation has been attained. The latter situation is deemed to exist if the specimen dilates on freezing. If no component of the concrete is critically saturated, the specimen should contract on cooling even through the freezing point. Concrete which can withstand soaking for a time equal to the freezing season without becoming critically saturated is considered immune to frost action in the particular environment under consideration.

7. *Abrasion resistance (coarse aggregate)* — Tests for abrasion resistance measure degradation caused by a combination of impact and surface abrasion. The test provides an indication of probable breakage in handling, stockpiling, and mixing. It is widely used as an index of aggregate quality and provides some indication of strength-producing potential. Although many specifications contain numerical limits, it should be noted that minerals differ in their relationship between abrasion resistance and strength, and crushed material abrades differently than rounded aggregate.

a. Los Angeles (rattler) machine (ASTM C 131, AASHO T96, ASA A37.7)

b. Deval machine (ASTM D 289)

Both these tests evaluate abrasion resistance from the increase in fineness produced by tumbling the aggregate with steel balls inside a steel vessel. The Los Angeles apparatus is by far the more widely used. It is a more discriminating test and requires less sample preparation than the Deval machine. By determining percentage of wear on a single sample after two different periods of exposure the Los Angeles rattler can be used to detect the presence in the sample of constituents that are markedly nonresistant to abrasion. A uniform material yields percentage of wear at a uniform rate whereas a sample containing a component that is markedly nonresistant to abrasion yields percentage of wear rapidly at first but with a diminishing rate as the test progresses.[49]

8. *Cleanness and deleterious substances*

a. Amount of material finer than the No. 200 sieve (ASTM C 117, ASA A37.4, AASHO T11). In this test the coarse aggregate is washed vigorously in water, and the wash water containing the fine material in suspension is passed over a No. 200 sieve.

b. Test for clay lumps in coarse aggregate (ASTM C 142, ASA 3728, AASHO T112). Material which can be pulverized by the

fingers is hand pulverized and separated from the sample by sieving.

c. Test for lightweight pieces (ASTM C 123). Aggregate deposits in which poor performance is associated with a minor lightweight fraction are usable when the light particles are removed by beneficiation or are present in small quantities. A limit is frequently placed on the permissible amount of light material when such deposits are used. To test for the presence and amount of such particles the sample is placed in a liquid having a specific gravity between that of the predominant component of the aggregate and the lightweight constituents whose presence is to be detected. The light materials float on the liquid and can be removed.

d. Test for organic impurities in sand (ASTM C 40, ASA A37.19). Organic materials may interfere with normal cement hydration. Specifications usually require that fine aggregate be free of injurious amounts of organic impurities. To perform the test the sample is placed in a sodium hydroxide solution and after 24 hr the color of the supernatant liquid is observed.

e. Sand equivalent test, test of the California Division of Highways. A sample of sand is agitated in a weak calcium chloride solution and the relative volumes of sand and flocculated clay determined. The test provides information on both the amount and the activity of the clay.

f. Alkali-aggregate reactivity. This is a special problem which does not influence specifications except in those areas where the problem is known to exist. In recognizing the problem, the service record of concrete made from any particular aggregate is especially important. If this service record shows cases of excessive cracking, or if no service record is available, it is suggested that a petrographic examination be made of the aggregate. The following laboratory tests are available.

(1) Quick chemical test (ASTM C 289). This has the advantage that it can be performed in 3 days, but for many aggregates the results are not conclusive.

(2) Mortar bar test (ASTM C 227). This test, in which the expansion of mortar bars stored over water is measured, is more conclusive but has the disadvantage of requiring several months and of requiring that coarse aggregate be crushed rather than tested in its normal state. With larger specimens, however, uncrushed aggregate may be tested.[50]

9. *Hardness (coarse aggregate)*

a. Scratch test (ASTM C 235). This test, intended to identify materials that are soft, including those which are so poorly bonded that the separate particles in the piece are easily detached from the mass, occurs frequently in specifications. Particles fail if they are scratched by a pointed brass rod of specified properties under a specified load.

b. Dorry hardness machine.[51] A rock cylinder is subjected to surface wear by finely crushed quartz on a revolving metal table.

c. Shore scleroscope.[52] Hardness is determined by the rebound of a diamond-tipped hammer dropped vertically on the test surface. This test is not widely used for mineral aggregates.

Properties not normally specified or tests infrequently performed

Most of the following tests have not been standardized.

1. *Toughness* — Falling hammer impact test.
   a. Rock cylinders (ASTM D 3, ASA A37.73, AASHO T5-35).
   b. Individual pieces of aggregate.[53]
2. *Compressive strength*
   a. Cylinder or prism compressive tests.[54]
   b. Crushing loose aggregate in a steel cylinder,[55] British Standard 812.
3. *Modulus of elasticity and Poisson's ratio*
   a. Compressometer tests of cylinders or prisms.[56]
   b. Tests of cylinder or prisms in which aggregate particles are included in a matrix of known properties.
4. *Particle shape*[34]
   a. Determination of dimensional ratios by proportional calipers, U.S. Corps of Engineers CRD C 119.
   b. Determination of void content.[57]
5. *Surface texture*[34] — Replica surface analyzer.[58]
6. *Porosity*
   a. Calculated from true and bulk specific gravities (ASTM C 127, C 128; ASA 27.5; AASHO T84, 785).
   b. Porosimeter.[59]
7. *Pore structure*
   a. Microscopic.[60]
   b. Adsorption.[61]
   c. Porosimeter.[62]
8. *Permeability* — Direct measurement of liquid or gas flowing through a specimen.[63]
9. *Specific heat* — Method of mixtures, U.S. Corps of Engineers CRD C-124.
10. *Thermal diffusivity*
    a. Calculated from specific heat, density, and conductivity.[41]
    b. Heating a solid block and measuring surface and interior temperatures.[64]
11. *Coefficient of thermal expansion*
    a. Optical lever extensometer.[4]

b. Resistance strain gage mounted on coarse aggregate. (U.S. Corps of Engineers CRD C 125).

c. Dilatometer.[65]

d. Mortar bar for fine aggregate (U.S. Corps of Engineers CRD C 126).

Visual observation and petrographic examination (ASTM C 295)

Visual observation can be a valuable addition to laboratory testing. A geologist can identify types of rock, formations in which they occur, and possible sources of trouble in a deposit. Even one not trained in geology can learn much by observing laboratory test specimens at the conclusion of tests such as those for compressive and flexural strength, soundness, and resistance to freezing and thawing.

A logical extension of this basic visual process, on which increasing emphasis is being placed in selecting concrete materials, is petrographic examination. The latter is defined as visual examination and analysis of the material in terms of both lithology and properties of the individual particles.[66,67] The procedure commonly uses a hand lens and petrographic and stereoscopic microscopes, but it can employ other techniques such as x-ray diffraction and differential thermal analysis.

Petrographic examination can quickly supply much pertinent information on the properties mentioned in preceding sections. It also assists in the interpretation of tests. It is the best available method for determining the presence of aggregate constituents capable of deleterious alkali reactivity and those containing swelling clays; it is also a good method for classifying texture (grain size) and structure (grain interlock) in aggregate constituents. Because of the nature of the examination, sample sizes are necessarily small. A large amount of work, therefore, is required for quantitative analysis of high precision. Much of the information is obtainable in an entirely objective fashion. Some of it, however, requires more personal interpretation than is required for most laboratory tests.

Petrographic studies commonly consist of two parts: (1) identification of the material or component; (2) an attempt to predict performance from past records of similar materials in service or from theoretical considerations. The method has found useful application in large organizations and has great potential, especially as an integral part of a well-equipped laboratory. By petrographic examination, valuable lessons of the past can be applied more tangibly to current problems provided that the pertinent data were originally well documented. However, there is a great need for more good service records of specifically identified materials under various conditions. Service records on materials of apparent similarity are not, however, necessarily dependable indices of performance unless restricted to the same geological areas.[68] In any case, long experience is required to be able to interpret, in terms of

service, data obtained by petrographic examination. It should be emphasized that specialized experience on the part of the petrographer is mandatory and close liaison with the concrete technologist desirable.

PART III — FEATURES OF PROCESSING AND HANDLING WHICH HAVE A BEARING ON CONCRETE QUALITY AND UNIFORMITY

Basic physical and chemical characteristics of aggregate cannot be altered by processing, although the quantities of certain deleterious particles can be reduced. Preparation and handling affect such important aggregate properties as gradation; uniformity of moisture content; cleanness; and, in the case of crushed aggregate, particle shape; thereby having an important influence on concrete quality. Economic factors will determine the degree to which processing can be carried in an effort to achieve desirable properties. Attention is directed to the appended list of references to the literature.

Aggregate processing may be divided into two broad classifications: (1) basic processing to achieve, principally, suitable gradation and cleanness; (2) beneficiation, to remove deleterious constituents.

Basic processing

Processes typically employed to provide aggregate of satisfactory grading and cleanness include the following:

1. Crushing and grinding — Stone, slag, and large gravel require crushing to provide the required distribution of sizes. Grinding is sometimes employed to produce sand sizes. In crushing gravel to produce angularity, rather than for necessary size reduction, quantities of desired large or intermediate fractions may be reduced and amounts of finer sizes unduly increased. There is also the possibility of creating undesirable particle shapes which may detract from workability or increase water requirement of concrete. As discussed below, crushing may have a by-product advantage for some sources by reducing the amount of soft and friable particles.

2. Screening — Screening is the primary process for producing desired gradation in the coarse aggregate size range. Although dry screening of crushed stone and blast furnace slag is quite common, it may be necessary at times to apply water during the screening process to remove fine particles which may be present in the material. In the case of gravel, screening is usually accompanied by the application of water to wash the material and expedite the separation of sand from coarse sizes. Normally, screening is employed only on sizes larger than the No. 8 sieve, although there are exceptions.

3. Washing — Washing is done to remove silt, clay, and excess fine sand. It usually begins with application of water during screening and is completed with removal of the unwanted fines in the overflow from

water classification. If the aggregate contains clay, mud, mudballs, or organic impurities in such quantities or so firmly attached that ordinary washing will not clean it adequately, scrubbers or log-washers may be needed. Taggart[69] describes the available processes.

4. *Water classification*—Sizing and control of gradation in the finer sizes are usually accomplished by classification in water. A wide variety of classifying devices are used for this purpose, all of which are based on different settling rates of different sized particles. Water classification is not feasible for sizes larger than about ¼ in. Sizing is not so sharp as in screening. However, the gradation can be controlled with considerable accuracy by suitable reblending, in spite of the overlapping in sizes.

Beneficiation

"Beneficiation" is a term used in the mining industry to describe the improvement in quality of a material by the removal of unwanted fractions. Success of a process depends on significant differences in the physical properties of desirable and undesirable constituents, such as hardness, density, and elasticity. The method to be employed, therefore, if any will be practicable, depends on the nature of the individual deposit. Processes which have been used with variable amounts of success include the following.

1. *Crushing*—Crushing may be used to reduce the quantities of soft and friable particles in coarse aggregates. Certain impact crushers are particularly adaptable to "selective" crushing. The degraded material is eliminated either by screening or by classification. The costs of installation and operation are likely to be high and there is always loss of sound material, which is frequently excessive; and removal of the degraded fractions may be difficult or expensive. On the other hand, in the case of many deposits, selective crushing is the only process available to make the material suitable for use.

2. *Specific gravity separation*—In many deposits, the deleterious fractions are of significantly lower specific gravity than the better quality parent material. Advantage is taken of this characteristic in several beneficiation processes.

a. High velocity water and air. Light materials, such as wood, miscellaneous trash, and some lignite, may be removed in a rapidly moving stream of water which carries the floatable material while allowing the heavier aggregate to sink. Some "trash" removers employ a vortex of water which throws the heavy particles to the periphery while allowing the light materials to rise for removal from the surface. Only large differences in specific gravity will yield efficient separation. High velocity air has also been used for this purpose in a few instances.

b. Jigs. They may be used to separate materials with much smaller differences in specific gravity than that required for high velocity water separation. Lightweight shales and cherts are prominent examples of materials which can be removed with good success. A jig is essentially a box with a perforated bottom in which a separating layer is formed by a pulsating water current. In certain types of jigs, even the finest sizes of coarse aggregate may be treated effectively. The overlap in specific gravity range common to both the rejected and accepted fractions can be maintained at a reasonably low figure where the procedure is applicable. Taggart[69] describes a variety of jigs and their operation (see also References 70, 71, and 72).

c. Heavy media separation. Water suspensions of heavy minerals may be used to separate materials of different specific gravities—the difference required being substantially less than required for successful jigging. The heavier fractions sink and the lighter material floats. The medium commonly used is a suspension in water of magnetite or magnetic ferrosilicon, substances which are susceptible to recovery by magnetic separators for re-use. By close control of the suspension, minimizing of turbulence, and elimination of fine particles (below about the No. 8 sieve), accurate elimination of material below a selected specific gravity level can be made. Removal of unwanted material of high specific gravity could also be accomplished by this process, although the need for such a procedure is rare. As in the case of other beneficiation methods, the practical applicability of heavy media separation depends on the nature of the parent aggregate, the degree to which specific gravity distinguishes between good and bad particles, and the requirements of the concrete for which the aggregate is intended.[70,72,73]

3. *Elastic fractionation*—Elastic fractionation is a recently developed process of extremely limited applicability. It involves dropping the aggregate particles onto a steel plate from which those with higher modulus of elasticity bounce farther than the presumably less desirable particles of lower elasticity. A separation is achieved by collecting the portions of differing rebound in separate compartments. There are several serious limitations to the method: (1) the shape of all particles must be somewhat similar, so as not to unduly affect rebound; (2) the process is not applicable to crushed aggregates because sharp edges interfere with elastic rebound; and (3) certain deleterious rocks, such as some of the harmful varieties of chert, have good elastic properties and are not eliminated. Elastic fractionation has been found to be applicable in only a few cases. As of the date of preparation of this report the committee knows of only one full-scale installation—and that is coupled with heavy media separation.[73]

4. *Magnetic separation*—The removal of iron from blast furnace slag with electromagnets is highly effective when the iron is in metallic form and has been liberated from the surrounding slag by crushing or grinding.

For general information on processing, the reader is referred to papers by Goldbeck,[74] Hubbard,[75] and Walker[76] and to the bibliographies contained in those papers. The art of processing is developing rapidly and current articles in the trade press keep abreast of improvements. Detailed information on many of these processes will be found in *Handbook of Mineral Dressing* by Taggart.[69] A general discussion of production and manufacture of aggregates is given by Rockwood.[77]

Handling of aggregates

Much can be done in the handling of aggregates to increase the likelihood of good performance. Conversely, basically good material may yield inferior results due to abuse in handling. Procedures for maintaining uniformity of gradation and moisture content are discussed in Section II of "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)."[78] Fig. 1 from that report is reproduced here and the principal recommendations which pertain to aggregates are given in abbreviated form.

1. Segregation in coarse aggregate is minimized when it is separated into appropriate individual size fractions to be batched separately.

2. Undersize smaller than the designated minimum size in each fraction should be held to a practical minimum, always less than the amount permitted by the specifications, and should be uniform in amount, particularly in the smaller sizes.

3. Stockpiles should be built in horizontal or gently sloping layers. Conical stockpiles or any unloading procedure involving the dumping of aggregates down sloping sides of piles should be avoided.

4. Trucks and bulldozers should be kept off stockpiles as they cause breakage and contamination.

5. Effective measures should be taken to insure accurate separation of sizes within specification limits as the aggregate is deposited in the batch plant. This can be accomplished by finish screening or by careful attention to stockpiling practices such as providing adequate separation between stockpiles and operating cranes to avoid swinging buckets of one aggregate over another.

6. Storage bins should have the smallest practicable horizontal cross section; the bottoms should slope at an angle not less than 50 deg from the horizontal toward a center outlet; and they should be filled by material falling vertically over the outlet.

7. Two sizes of sand cannot be blended satisfactorily by placing them alternately in stockpiles, cars, or trucks. Blending, if required to improve grading, should be done by feeding the different sizes into a common stream from regulating feeders onto the belt or loader. Where two or more sizes of sand are employed, it is preferable to batch them separately.

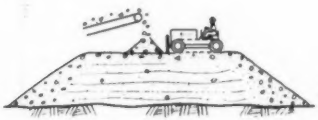
INCORRECT METHODS OF STOCKPILING AGGREGATES CAUSE SEGREGATION AND BREAKAGE

PREFERABLE

Crane or other means of placing material in pile in units not larger than a truck load which remain where placed and do not run down slopes.

OBJECTIONABLE

Methods which permit the aggregate to roll down the slope as it is added to the pile, or permit hauling equipment to operate over the same level repeatedly.

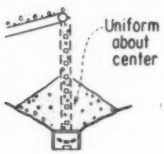


LIMITED ACCEPTABILITY—GENERALLY OBJECTIONABLE

Pile built radially in horizontal layers by bulldozer working from materials as dropped from conveyor belt. A rock ladder may be needed in this setup.

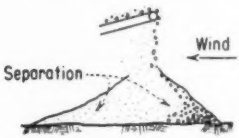
Bulldozer stacking progressive layers on slope not flatter than 3:1. Unless materials strongly resist breakage, these methods are also objectionable.

STOCKPILING OF COARSE AGGREGATE WHEN PERMITTED
(STOCKPILED AGGREGATE SHOULD BE FINISH SCREENED AT BATCH PLANT; WHEN THIS IS DONE NO RESTRICTIONS ON STOCKPILING ARE REQUIRED)



CORRECT

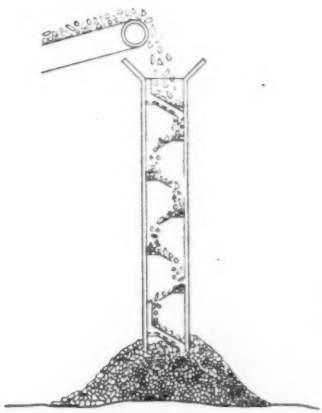
Chimney surrounding material falling from end of conveyor belt to prevent wind from separating fine and coarse materials. Openings provided as required to discharge materials at various elevations on the pile.



INCORRECT

Free fall of material from high end of stacker permitting wind to separate fine from coarse material.

UNFINISHED OR FINE AGGREGATE STORAGE (DRY MATERIALS)



When stockpiling large sized aggregates from elevated conveyors, breakage is minimized by use of a rock ladder.

FINISHED AGGREGATE STORAGE

Fig. 1—Correct and incorrect handling of Aggregates. (Reproduced from ACI 614-59, Reference 78)

8. Wind should not be permitted to segregate dry sand.

9. To the extent practicable, wet sand should be drained until it reaches a uniform moisture content. Generally a satisfactory and uniform stable condition will be reached in about 48 hr or less.

Control of particle shape

The particle shape of crushed aggregates is dependent on the crushing equipment used. A universal guide cannot be written because it has been found from experience that equipment which produces acceptable particle shape with one type of rock will not necessarily produce acceptable shape with another type. This is a problem for the producer which is mentioned here so that the user may be aware of it. Normally he will be able to observe the particle shape before he accepts the aggregate. In some cases, particularly on large projects, the user may be in a position to do some experimenting with the material to determine in advance what sort of particle shape can be feasibly produced.

PART IV — SELECTING AGGREGATES

A high quality aggregate consists of particles which are free from fractures, not easily abraded, favorably graded, and not flat or elongated; which do not slake when wetted and dried, whose surface texture is relatively rough with little or no unfavorable capillary absorption and which contain no minerals that interfere with cement hydration or react with cement hydration products to produce excessive expansion. Other properties are not so easily specified; for instance, thermal conductivity should be high if the chief concern is getting the heat out of the interior of a dam, and low in a building wall where insulating value is important.

The ideal aggregate is seldom available. The problem is to decide the level of performance that is required in a given situation and determine the degree to which it is economically attainable. It is necessary to appraise the available aggregates. All tests have limitations which make them not completely reliable; therefore, the service record, if available and properly interpreted, becomes a valuable source of information.

Since the causes of deterioration of concrete are many and even experts are often in disagreement in a given instance, there is a great risk of wrongfully condemning an aggregate on the basis of its presence in defective concrete. Records must be dependable and the evidence of a recurring nature before rejection is made. However, popouts provide a dependable criterion of identifying an undesirable component in an aggregate. A structure completely sound after 10 years or more representative service can be assumed to constitute an "endorsement" of all materials used in it, including the aggregates. There

is the possibility, however, that potentially reactive aggregates may have a satisfactory service record if they have been used only with low-alkali cements. Furthermore, it is important to check that the quality of the material currently available from the source is at least equal to that used in the structure of good record. Where deterioration is associated with a minor constitutent, beneficiation may make a previously undesirable aggregate usable. Close visual observation is an all-important aid to good judgment. Where the service of a petrographer is available, the appraisal of an aggregate's service record can be made on a much more scientific basis. The petrographic examination provides information on a wide variety of properties and frequently makes it possible to compare samples from a newly developed aggregate source with others of known service record.

In selecting an aggregate it is economical to require only those properties pertinent to its use in a particular project. As a general guide the following criteria are suggested.

a. Regardless of use, the grading of the aggregate should be uniform throughout its period of use and should conform to some reasonable grading requirements. Excellent concrete can be made with aggregates differing quite widely in grading characteristics so long as they remain within the tolerances of usual specifications, for example, ASTM C 33. Actually, good concrete can be made with some aggregates outside these limits, including those showing discontinuous gradings, if enough care is taken in proportioning concrete mixtures[45] to determine optimum proportions. However, unless the job is large enough to justify the experimentation needed to establish effective mix proportions, or the grading of the particular aggregate is known to produce satisfactory concrete, aggregates meeting standard grading specifications should be used. In any case, it is important that once the grading is established, it should be maintained constant within rather close tolerances. Otherwise effective job control is impossible.

If after establishing optimum proportions for fine aggregate, coarse aggregate, and cement, the fineness of the sand or the percentage of undersize in the coarse aggregate increase, the water requirement for the required slump will be raised. If water is added without compensating adjustments in cement, strength and durability will be reduced. A decrease in the fineness of the sand can adversely affect the workability of the concrete by introducing harshness. Fluctuations in fineness also introduce an added difficulty in achieving uniform air entrainment. The abrasion resistance of the aggregate influences the stability of the gradation during handling and mixing.

b. An aggregate with unfavorable particle shape should not necessarily be rejected in favor of a more expensive aggregate with better

particle shape if the cost of additional cement required for the first aggregate is less than the extra cost of the second aggregate, and the use of the additional cement will not be detrimental.

c. An aggregate that is contaminated with organic material to such an extent that the contamination interferes materially with the setting of the cement should not be used.

d. An aggregate that will not produce concrete of the required strength should not be used. If required strength can be achieved only with an excessively high cement factor, use of the aggregate is probably uneconomical.

e. A material to be used in concrete to be exposed to severe freezing and thawing should be shown to be capable of producing frost resistant concrete either by service record or freezing-and-thawing tests.

f. A material to be used in concrete exposed to severe weathering that should also maintain a defect-free appearance should be essentially free of particles that are soft or friable, that have unfavorable capillary absorption, or that will weather to produce staining.

g. A material containing or consisting of substances that could react with alkalies in the cement to cause excessive expansion should not be approved for use in concrete that will be exposed to wetting unless it is also required that low-alkali cement is used or that an adequate quantity of a suitable pozzolan is used, or both.

h. In unusual circumstances the aggregate user may wish to obtain materials with particular thermal or elastic properties; in such cases he may expect to pay a premium for the aggregate.

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This report was submitted to letter ballot of the committee which consisted of 13 members; 12 members returned their ballots, all of whom voted affirmatively.

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Discussion of this report should reach ACI headquarters in triplicate by Feb. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Selección y Uso de Agregados para Hormigón

Presentación de información disponible para los agregados en cuatro categorías:

(1) Evaluación de las propiedades del agregado en relación a su influencia sobre las propiedades del hormigón.

(2) Métodos para determinar las propiedades de los agregados y las limitaciones de dichos métodos.

(3) Rasgos de la manipulación y preparación del agregado que tienen relación con la calidad y uniformidad del hormigón.

(4) Selección del agregado.

Este informe se limita solo a la arena, grava, piedra molida y escorias de altos hornos enfriadas al aire. Los agregados ligero y especial pesado no están comprendidos en el mismo.

Le Choix et l'Emploi d'Agrégats pour Béton

Présentation des renseignements disponibles sur les agrégats dans quatre catégories. (1) Evaluation des propriétés des agrégats selon leur influence sur les propriétés du béton. (2) Méthodes de détermination des propriétés des agrégats et les limitations de ces méthodes. (3) Les particularités de la préparation et la manutention des agrégats qui ont une influence sur l'uniformité et la qualité du béton. (4) Choix de l'agrégat.

L'exposé se borne au sable, au gravier, à la pierre cassée et à l'agrégat de laitiers de hauts-fourneaux refroidis à l'air. Les agrégats légers et les agrégats lourds spéciaux ne sont pas dicutés.

Auswahl und Anwendung von Zuschlagstoffen für Beton

Es werden die vorhandenen Angaben über Zuschlagstoffe in 4 Gruppen aufgefuehrt.

(1) Bewertung der Eigenschaften der Zuschlagstoffe im Hinblick auf ihren Einfluss auf die Eigenschaften des Betons.

(2) Methoden, um die Eigenschaften der Zuschlagstoffe zu ermitteln und die Beschränkungen dieser Methoden.

(3) Besonderheiten der Aufbereitung und Behandlung von Zuschlagstoffen, die einen Einfluss auf die Güte und Gleichförmigkeit des Betons haben.

(4) Auswahl von Zuschlagstoffen.

Der Bericht ist auf Sand, Kies, zerkleinertem Stein und luftgekühlter Hochofenschlacke begrenzt. Leichtzuschlagstoffe und besondere Schwerzuschlagstoffe werden nicht behandelt.

Title No. 58-25

Precast Complex Conoidal Horticultural Domes

By W. JOHN HUFSCHMIDT

A radical departure from the standard gable type roof design for greenhouses or horticultural exposition buildings was undertaken by the Milwaukee County Park Commission at Mitchell Park in Milwaukee, Wis. There were a number of innovations in design, both from the horticultural, as well as the architectural-structural, point of view.

Describes the design, casting, and erection of one of the typical superstructures of these complex conoidals, or domes, as they are more commonly called.

An interesting aspect was the precasting of the dome sections. The use of concrete molds with loose concrete pieces or cores was selected. Because the large pieces were not in one plane, the molds required loose pieces which had to be released before the finished product could be stripped or removed. The construction of the concrete mold, which followed much the same procedure as used in the foundry industry, is described.

■ THREE COMPLEX CONOIDALS, or domes as they are more commonly called, are well along toward completion as horticultural exhibition buildings in conjunction with the new $3½ million Botanical Garden Development undertaken by the Milwaukee County Park Commission. The three domes of precast concrete sections are similar in design but have separate functions: tropical, arid, and temperature horticultural exhibition buildings. For an artist's conception of the finished project, see Fig. 1.

DESIGN CRITERIA

The basic design criteria were determined by the horticultural function of each dome. In both the tropical and arid domes, a maximum amount of sunlight is required to produce the best possible examples of full sized plants, such as rubber trees, palm trees, and banana trees as well as various species of cacti. To provide necessary sunlight at the Milwaukee latitude (43 deg north), and considering the many days the sun does not shine during the winter months, it became necessary to devise a method of concentrating the sun's rays so that full sized

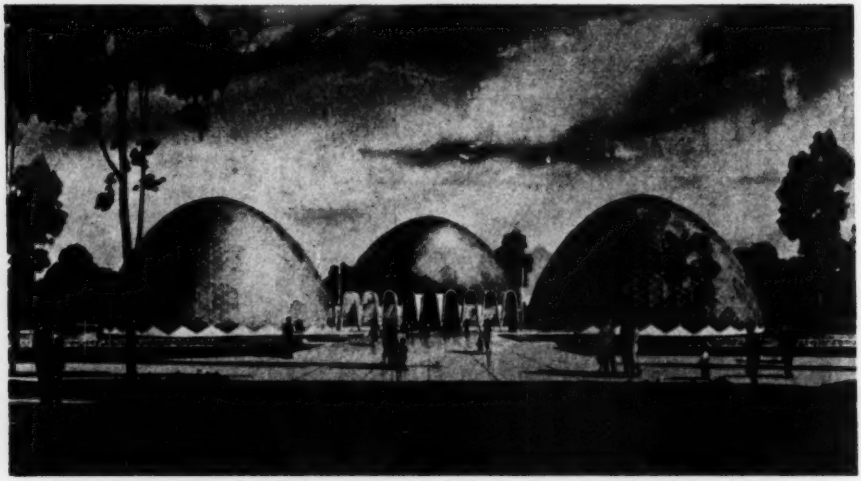

Fig. 1—An artist's rendering of the three domes of Milwaukee's Botanical Garden Development at Mitchell Park

tropical and arid trees and plants be supplied with the necessary ultraviolet rays of the sun's spectrum so vitally needed for photosynthesis in plant growth.

A second, and equally important, requirement in the design of the structure dictated that the inside of the dome be free from high humidity and condensation to present the most inviting atmospheric conditions for the viewing public.

With these two basic considerations set before him, the architect, Donald L. Grieb of Milwaukee, laid out the three complex conoidal domes and proportioned and grouped them with their connecting foyers in an outstanding and revolutionary approach to the public display of horticultural beauty. The basic configuration of the domes is copyrighted.

STRUCTURAL ENGINEERING

The firm of Ammann & Whitney, New York and Milwaukee, were employed by the architect to furnish the necessary structural engineering studies and working drawings. Robert Hopwood, manager and chief engineer of the Milwaukee office, describes the work as follows: "Almost at the outset, we decided to use concrete on this structure as the most practical material to resist the attacks of moisture and insecticides from within the domes. The architect, having furnished us with the surface of revolution, made one other request which was to give them a surface that would permit 75 percent unobstructed light to enter.

ACI member **W. John Hufschmidt** is president and treasurer of Hufschmidt Engineering Co., Menomonee Falls, Wis. Hufschmidt Engineering Co., bridge contractors and general contractors, have also specialized in prestressed concrete, precast concrete stadiums, vacuum concrete, and the producing and erection of various other precast concrete units.

Ordinary smooth domes are not difficult to design. We began the design as a smooth surfaced dome, then switched to a patterned surface, much the same as one would do in the design of a truss to replace a beam design. The principal stresses in a dome are compression along the meridians and either compression or tension along the horizontal circles.

"The first step was to find what patterns of members were suitable to carry the resolved stresses and maintain stability in the structure. These patterns were discussed with the architect for his final choice in beginning his aesthetic treatment of the structure. The dome stresses were finally grouped at juncture points and resolved into the individual members.

"Due to the large cost of the form work, it was decided to detail the dome for precast handling by selecting field joints to create the smallest number of members and at the same time provide the greatest re-use of forms."

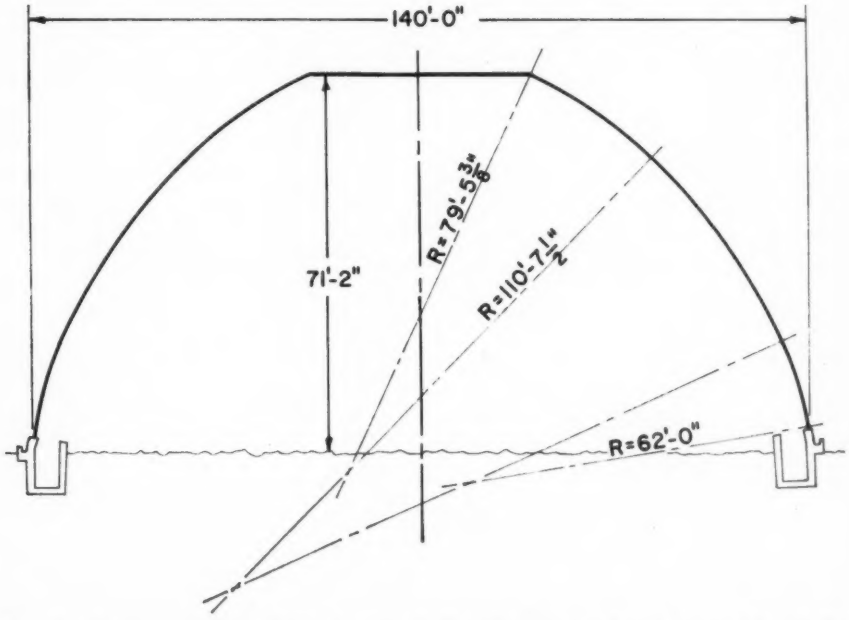

Fig. 2—A cross section through one of the domes showing the profile of the curves used and the dome's dimensions

The design was based on a 30 lb per sq ft vertical live load component. Strength of the concrete was limited to 5000 psi and the plastic or ultimate theory was used in the final design.

GEOMETRY

Fig. 2 shows a typical section through one of the domes. Fig. 3 shows the six basic geometrical shapes that comprise one section of the dome. From these two figures the individual pieces were detailed. Fig. 4 shows a drawing of Part 6, the largest, of which 25 pieces were required per dome. Due to their size, 18 x 18 ft, their several points of elevation had to be located in relation to a given plane.

This required computing exact and relatively different elevations at Points A, B-C, D, E-F, and G for Part 6. Likewise, Line BE is not parallel to Line CF, because each segment is converging as it proceeds up the face of the dome. This results in mathematical computation in both the vertical and horizontal direction. These calculations were used to build the forms and to establish a set of coordinates for erection purposes.

PRECAST MANUFACTURING

The architect recognized that the project was for a public body and therefore would necessarily have to be competitively bid. He further realized that end result type specifications would be the most practical because of the complex nature of the castings and erection of the domes. He therefore showed the four basic radii of the vertical projection and the six unit pieces making up one segment of the 25 segmented dome. The method of casting and erection, while all subject to his approval before proceedings, was left entirely to the successful low bidding precast contractor.

This freedom of manufacture allowed the precast manufacturer maximum use of his ingenuity in design of forms, casting of pieces, and method of temporary falsework supports.

BUILDING THE FORMS

To arrive at a competitive price, the kind of forms to be used (wood, metal, or concrete) had to be thoroughly studied. The most important

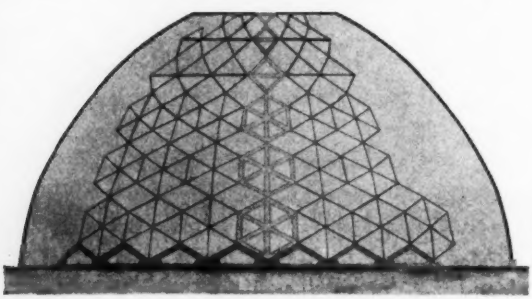

Fig. 3—The sketch shows the six basic geometrical shapes that comprise one section of the dome

of the requirements were: (1) a minimum number of joint or parting lines; (2) pieces that strip easily; (3) uniformity of size and shape; (4) 75 castings per form (25 segments per dome); (5) forms exposed to weather through a 2 year casting cycle.

When all requirements were analyzed, the use of concrete molds with loose concrete pieces or cores was selected. Because the large pieces were not in one plane, the molds would require loose pieces which had to be released before the finished product could be stripped or removed.

The construction of the concrete mold followed much the same procedure as used in the foundry industry. A master pattern of the piece was made. It consisted of a number of segments made of plaster of Paris, which were assembled in the final form on an inverted shaped concrete casting pallet. When these several sections had been set perfectly and the intersections joined and molded with plaster of Paris, a perfect finished piece was exposed in an upside down position resting

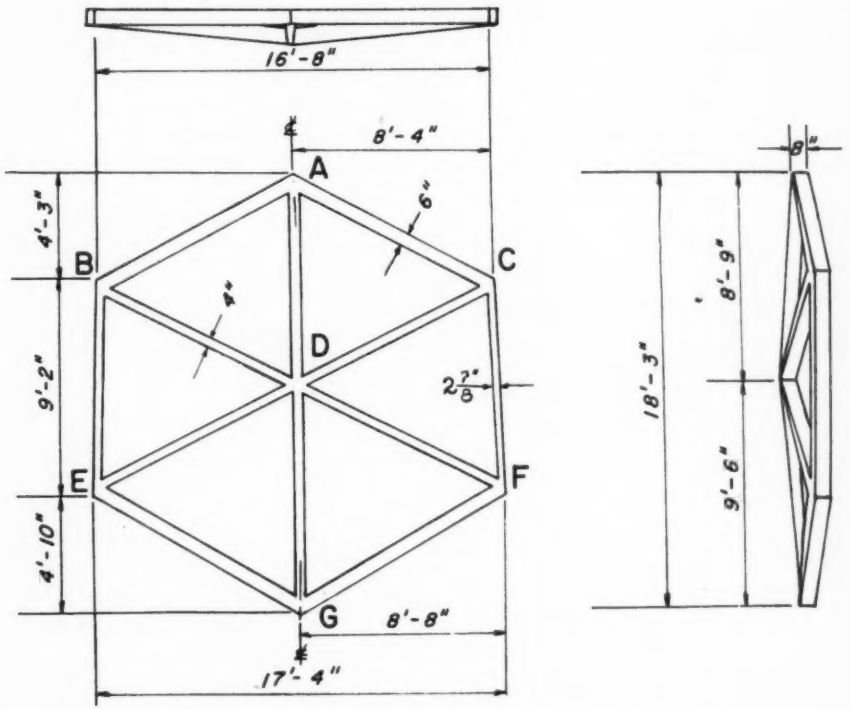

Fig. 4—This hexagonal shape is the largest precast piece of the dome (Piece 6) Twenty-five of these 5000 lb pieces were needed for each dome

on the concrete pallet (Fig. 5). A wood perimeter form was then placed around the finished plaster of Paris and reinforced concrete was cast. This concrete then became the finished mold.

After the mold had obtained its strength of 3750 psi, two 54 lb, 18-in. I-beam strong backs were bolted to pre-set anchor connections. The mold then weighed approximately 15 tons. It was rolled over and the I-beams became the base support. Next, the plaster of Paris master pattern was removed, and the mold was now almost ready to receive the casting of finished pieces. The remaining step was to cast concrete "loose pieces" which had to be removed each time prior to extracting a finished section, to allow draft or draw of the piece.

CASTING AND STRIPPING

Realizing the small average quantity of concrete required per daily placement (5 to 6 cu yd) and the slow rate of discharge into the molds, it was decided to make the concrete at the job site. A one bag concrete mixer was set up close to the forms and the concrete was deposited into the mold from wheelbarrows.

All aggregates were weighed and water accurately proportioned before being placed in the mixer. The production of any one dome was governed by the ability to produce Part 6 from one mold.

It was decided that two pieces of this largest section per week would be the minimum required. This meant that 13 weeks of casting would be required to produce Part 6, and therefore all the other sections would

Fig. 5—Plaster component parts being assembled to form the master pattern from which the concrete form was cast

likewise be cast simultaneously in this same approximate time. Two sections were cast every day and stripped in 3 days.

Stripping of a relatively thin section (2⅞ x 8 in.) of concrete such as this in 3 days, without the use of artificial heat or high-early-strength-cement, required an accurately controlled design mix. A number of trial mixes were made with the main object being to obtain 2500 psi concrete in 3 days. A cylinder testing machine was set up at the job site and all pieces had 3, 7, and 28 day cylinders cast for each day's placement.

The mix as determined by test was as follows: ¾ in. stone, 325 lb; sand, 195 lb; cement, 94 lb; slump, 3 in. None of the concrete tested less than 5000 psi at 28 days.

Curing was done with wet burlap during the hot months and canvas cover and hay during late fall. No concreting was done during the winter months. All cylinders were job-site cured so that the actual strength of the cylinders more nearly represented the concrete section to be used.

The molds were oiled with paraffin base form oil. When ready for removal, the pieces were stripped with the aid of a crane hooked to a six-point pickup bridle.

All reinforcing cages were tack welded together in jigs, so proper concrete coverage could be maintained in all directions. Steel inserts, which were to provide anchorage connections for the aluminum and stainless steel sash, were placed in the wet concrete with the help of positioning templates.

After the pieces were removed and stock piled, all surfaces were hand rubbed. In the finished dome they will be painted with three coats of special paint which will not support fungi growth. This is necessary because the fungi, if allowed to grow, may upset the atmospheric balance required for full exhibit plant growth.

ERECTION

Because each horizontal ring had a different bottom and top diameter and consisted of 25 sections per ring, it was decided that any form of temporary support would have to allow for angular adjustment. Further, the bottom sections, which are the heaviest pieces, weighed over 5000 lb each and the pieces of each succeeding ring decrease in weight and size. As the rings moved up, the vertical load increased. It was also recognized that the area inside the dome must be kept relatively clear so that the location of the pieces as they were erected could be easily checked each day. Finally, any falsework must be made in such a fashion that it could be disassembled and removed from the inside and later re-erected for the next dome.

From these facts, a falsework (Fig. 6) consisting of two circles of 8 in. pipe columns with 25 columns per ring was established as the main supports. These column rows were set on temporary cast concrete footings and supported at the top with two rows of 3 x 3 x ½-in. angles rolled in sections to the arc of their respective radii. Later X-bracing consisting of ⅜ in. steel cable and turnbuckles was provided to make each ring self-supporting. The column rings were joined together with 8-in. pipe to complete the main falsework base structure. The steel

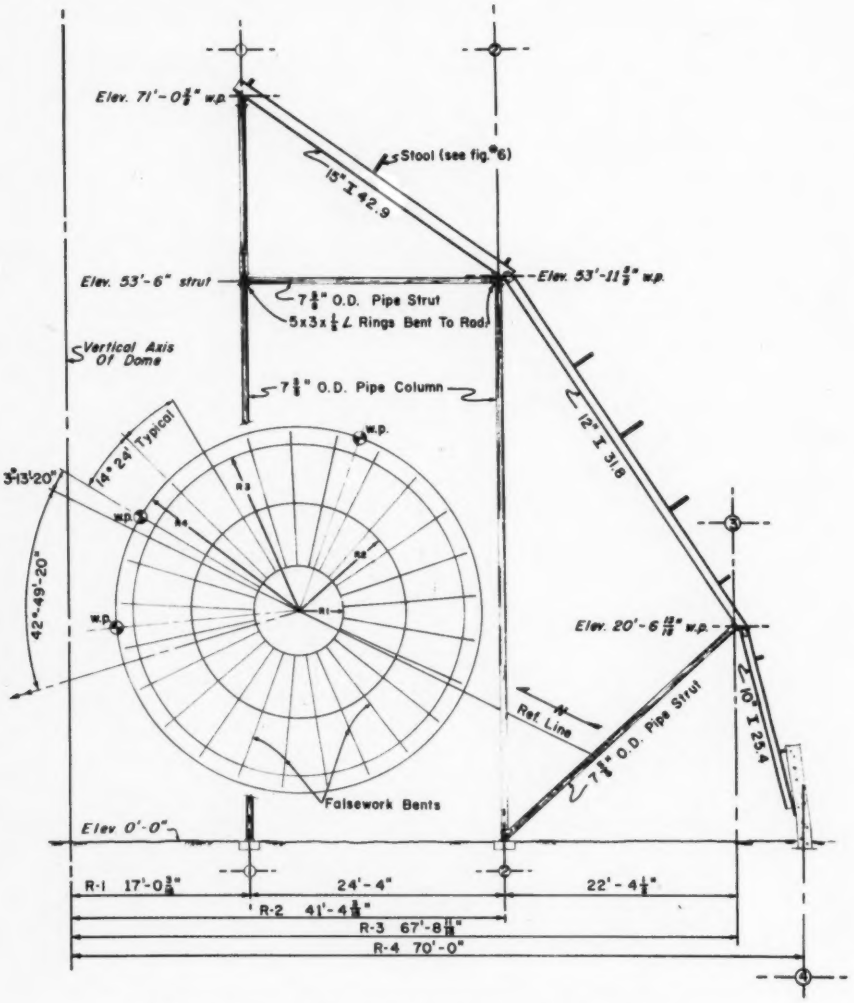

Fig. 6—Typical falsework details showing plan of bents

Fig. 7—Threaded adjustment legs on the falsework members permit adjustment of the precast members to proper coordinates

columns of the inner ring were 53 ft high, and an 18 ft splice-on piece was provided to reach the top. This splice was also needed to provide columns short enough to be removed from the inside of the dome.

ADJUSTMENT

To this framework of pipe columns, three sections of 12 in. I-beams were bolted. These beams receive the point load of the concrete dome. At the point of contact of the dome and steel support, a flat plate with two threaded adjustment legs was welded to the I-beams (Fig. 7). This adjustment allowed the perfect positioning of the dome pieces in relation to horizontal and vertical planes. To obtain these points, a set of accurately placed templates were used to give horizontal dimensions radially from the known center point.

Vertical distances were projected with a 20 lb plumb bob hanging from the piece by piano wire and swinging free in a pail of oil. This was necessary to correct for excessive height and resulting wind sway of the plumb bob.

The coordinate points are extremely important, because any movement of the dome pieces in or out from center would change its diameter. This must not happen or the so-called "keystone" piece (25th) in any ring would not fit.

As erection proceeded, it became more obvious that almost every piece in each ring must be checked on the coordinates or the ring would not close.

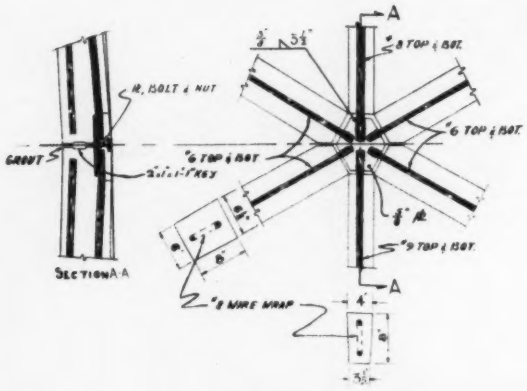

Fig. 8 — Typical connection details for precast sections

WELDING AND GROUTING

A typical connection joint and splice bar arrangement is shown in Fig. 8. A ⅜ in. fillet weld on each side of the reinforcing bars connected them to the ⅜ in. splice plate. All welding was done by certified welders in accordance with AWS specifications. At least 25 percent of all the welding at any one joint was completed before allowing the pieces to be freed from its crane support. The rest of the welding was completed as the next piece was being rigged for erection.

The grouting of the joint connections consisted of a three stage operation. First, hand pressure, gun grouting of the under or inner side of the joint was completed. Second ¼ in. of rich, semidry grout (1:3) mix was hand packed in all corners and around the weld plate. Third, the following day a finish coat of rich grout was applied, troweled off, and covered with curing compound.

This procedure prevented any excess shrinkage, and only in a few cases was it necessary to return and apply grout to the shrinkage cracks that had formed.

The structure was carefully inspected and the exact location of key points was accurately measured before backing off on the adjustment bolts, releasing the load from the falsework. After the falsework had been removed, another careful inspection was staged and the same key points were again checked. No noticeable movement could be detected.

CONCLUSIONS

The bidding specifications required that a tolerance of ± 1 in. at any point in the horizontal plane of any given ring diameter be maintained. To check this out, and to provide further information for the subsequent

sash and glazing contractor, a complete drawing of "as is" coordinates was required. This drawing showed that at no point did the diameter vary greater than ± ⅝ in.

The structural portion of each dome covers an area of over 30,000 sq ft. The contract price for the superstructure of concrete covering three domes was $271,000, or about $3 per sq ft of dome area. Any structure of a specialized nature such as this should not be related to more conventional buildings when comparing costs on any relative basis.

Of the three structural materials available today—wood, steel, and concrete—only concrete for all practical purposes can be molded to any desired shape as illustrated by these three complex conoidals and their barrel shaped connections.

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Cúpulas Complicadas de Hormigón Pre-Fundido en Forma de Cono para Horticultura

La Milwaukee County Park Conmission en Mitchell Park, Milwaukee, Wisconsin EE UU utilizó un diseño que se aparta radicalmente del tipo "Standard" de techo a dos aguas para invernaderos o edificios para exhibiciones de Horticultura. Se ve un número de innovaciones en el diseño, no sólo en la estructura y arquitectura, sino desde el punto de vista horticultural.

Se describe el diseño fundido y erección de una de las típicas super-estructuras de estos techos de forma de cono complicados o cupulas como se acostumbra llamarlos.

Un punto interesante fué la fabricación con elementos premoldeados de las secciones de las cupulas. Se prefirió y eligió usar moldes de hormigón con núcleo ó piezas sueltas. Como las piezas grandes no estaban en un plano, los moldes requirieron piezas sueltas que tenían que liberarse antes que el products final pudiera ser desmoldado o removido. También se describe la construcción del molde para el hormigón que es más o menos igual a la usada en la industria de fundición.

Dômes Horticoles Conoides Complexes, Préfabriquées

Une déviation fondamentale de la conception habituelle de comble sur pignons pour serre-chauds ou bâtiments d'exposition horticoles a été entreprise par le Milwaukee County Park Commission à Mitchell Park, Milwaukee, USA. Il y avait des innovations de conception tant du point de vue de l'horticulture que de l'architecture et de structure.

On décrit la conception, la coulée et le montage d'une superstructure-type de conoide complexe ou dôme, ainsi qu'on les dénomme plus communément.

Un aspect intéressant c'était la préfabriquée des sections du dôme. L'emploi de moules de béton ayant des parties ou de noyaux amovibles a été choisi. Puisque les grands éléments n'étaient pas d'une seule pièce, les moules nécessitaient des parties amovibles, enlevées avant que le produit fini puisse être mis à nu ou extrait. On décrit la construction du moule de béton, qui suivait la même marche qu'on emploi dans la fonderie.

Vorgegossene, zusammengesetzte, glockenförmige Gartenbaukuppeln

Eine vollständige Abwendung von der Standardausführung für Gewächshäuser und Gartenbau-Ausstellungsgebäuden wurde von de Milwaukee County Park Commision im Mitchell Park in Milwaukee, Wis., durchgeführt. Es gas eine Anzahl Neuerungen im Entwurf, sowohl vom gartenbaulichen als auch vom architektur-baulichen Gesichtspunkt aus.

Der Entwurf, Guss und die Errichtung einer der typischen Aufbauten dieser zusammengesetzten, glockenförmigen Kuppeln oder Dome, wie sie mehr allgemein genannt werden, wird beschrieben.

Ein interessanter Teil war das Vorgiessen der Kuppelteile. Die Benutzung von Betonformen mit losen Betonteilen oder -kernen wurde gewählt. Da die grossen Teile nicht in einer Ebene lagen, benötigten die Formen lose Teile, die entfernt werden mussten, ehe das Fertigteil von der Umhüllung befreit oder entfernt werden konnte. Die Konstruktion der Betonform, die sehr dem in der Giesserei-Industrie angewandten Verfahren ähnelt, wird beschrieben.

Title No. 58-26

Tie Requirements for Reinforced Concrete Columns

By B. BRESLER and P. H. GILBERT

Mechanism of failure of reinforced concrete columns with lateral ties is described. Rational criteria for spacing and size of lateral ties are derived, and pilot tests verifying some of these criteria are described.

■ PRESENT CRITERIA FOR THE DESIGN of tied columns and piers specify varying requirements for lateral ties. For example, the ACI Building Code (ACI 318-56) requires "lateral ties at least ¼ in. in diameter, spaced not over 16 bar diameters, 48 tie diameters, or the least dimension of the column. When more than four vertical bars are used, additional ties are to be provided so that every longitudinal bar is held firmly in its position and has lateral support equivalent to that provided by a 90-deg corner of a tie." This requirement does not consider the effect of the magnitude of the yield stress in the longitudinal reinforcement on the required spacing of ties, nor does it indicate when tie size should be greater than ¼ in., or allow tie size less than ¼ in.

Review of other codes reveal similar limitations in design criteria for ties in reinforced concrete columns.

An extensive review of technical literature[1-7] failed to disclose any analysis of the effect of lateral ties on the behavior of reinforced concrete columns. It has been suggested by several investigators of reinforced concrete columns that the role of lateral ties is to prevent buckling of longitudinal reinforcement, but no analytical formulation of the problem has been proposed.

A preliminary study of tests[1-7] resulted in the following hypothesis of the mechanism of failure:

1. As the axial load on the column increases, the local strains or stresses in the outer concrete cover increase until they reach a limiting value.

2. At this limiting strain or stress the exterior cover cracks, so that it can no longer support appreciable load, or it spalls off.

3. The loss of the cross-sectional area of the concrete cover imposes additional stresses on the remaining concrete core and the steel reinforcement.

4. The steel reinforcement under this additional stress begins to yield or buckles outward.

5. Loss of stiffness of the steel due to yielding and/or impending buckling causes additional strain in the concrete area.

6. The core, subdivided into short prisms by the ties, maintains its integrity until its ultimate strength is reached and then fails suddenly.

7. At this point the entire column fails and the load carrying capacity is lost. The above events occur rapidly and the failure is often characterized as occurring instantaneously.

Further study of the role of ties in reinforced concrete columns indicated that the size and spacing of ties influence the following factors:

1. Local state of stress (or strain) in the concrete in the zone adjacent to the ties.

2. Strength of the concrete core.

3. Buckling strength of longitudinal reinforcement, particularly when the concrete cover may be lost due to imposed load or accidental damage.

STRESS IN CONCRETE COVER

One of the primary reasons for cracking and spalling of the cover is the stress concentration produced at the interface between the lateral tie and the surrounding concrete. The state of stress in the concrete at this interface cannot be defined precisely, but the nature of local stresses and deformations in the shell due to presence of the ties may be seen in Fig. 1.

Longitudinal compression produces lateral expansion of the concrete due to the Poisson effect. Without lateral ties, such lateral expansion would occur freely and uniformly along the length of an axially loaded column. The lateral ties partially restrain such lateral expansion, and prevent displacement of longitudinal bars at the level of the ties, causing a distortion of the cross section as shown in Fig. 1. Stresses produced by this distortion are not analyzed here, but they undoubtedly contribute to the initial failure in the shell. These stresses and deformations in ties are now under study at the University of California at Berkeley, as well as in other laboratories.

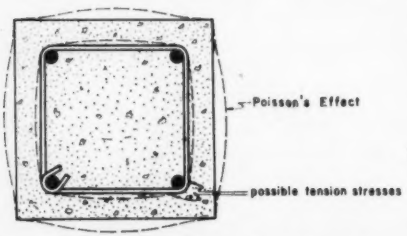


Fig. 1—Cross section of column showing distortion of the section at tie level

ACI member **Boris Bresler**, professor of civil engineering, University of California, Berkeley, was the co-awardee of the Wason Medal for research in 1959. Mr. Bresler is chairman of ACI-ASCE Committee 341, Reinforced Concrete Columns, and also a member of ACI-ASCE Committee 326, Shear and Diagonal Tension. He joined the faculty of the University of California in 1946.

P. H. Gilbert, a former graduate student at the University of California, is a lieutenant in the U. S. Army.

TIE SPACING AS GOVERNED BY STRENGTH OF CORE

Two modes of failure observed in early column tests correspond to the results obtained[2] with plain concrete specimens. Short compression specimens such as standard test cylinders are laterally restrained at the ends by the heads of the testing machine and fail in a typical "shear cone" failure, Fig. 2a. This type of failure is related to the state of stress resulting from the combination of the axial compression and the lateral restraint.

Long prisms subjected to compression, or short specimens with friction at the ends greatly reduced or eliminated, fail along planes parallel to the longitudinal axis, Fig. 2b. The strength corresponding to the latter mode of failure is about 20 percent lower than that for the first mode of failure. In columns with unduly large tie spacing, failure in the core may occur by longitudinal splitting as in a long prism of plain concrete. To obtain the maximum effectiveness of the concrete core lateral ties should be spaced so as to prevent failure by splitting. Using the results obtained in standard compression cylinders as a guide,

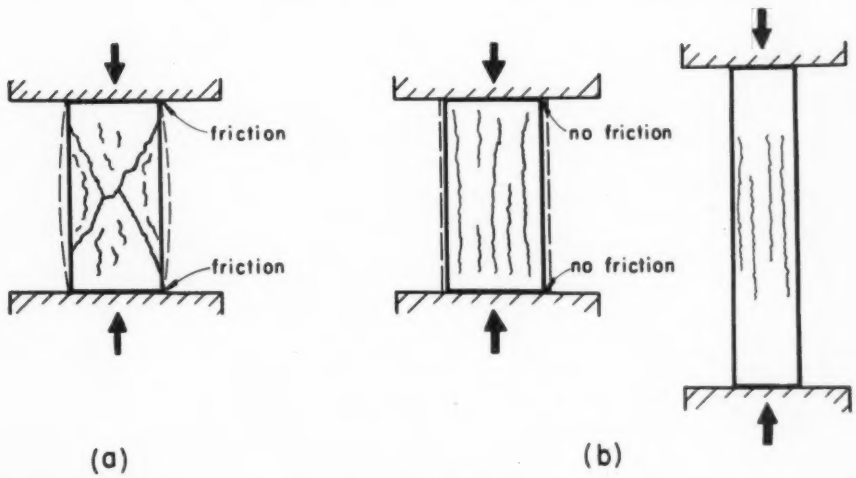


Fig. 2—Typical failure of plain prisms; (a) typical "shear cone" type failure, (b) typical "simple compression" or longitudinal splitting type failure

the ties should have a spacing equal to or less than twice the core dimension of the column. A convenient value for tie spacing which falls into this range is the least lateral dimension of the column, which is the value now provided in ACI 318-56.

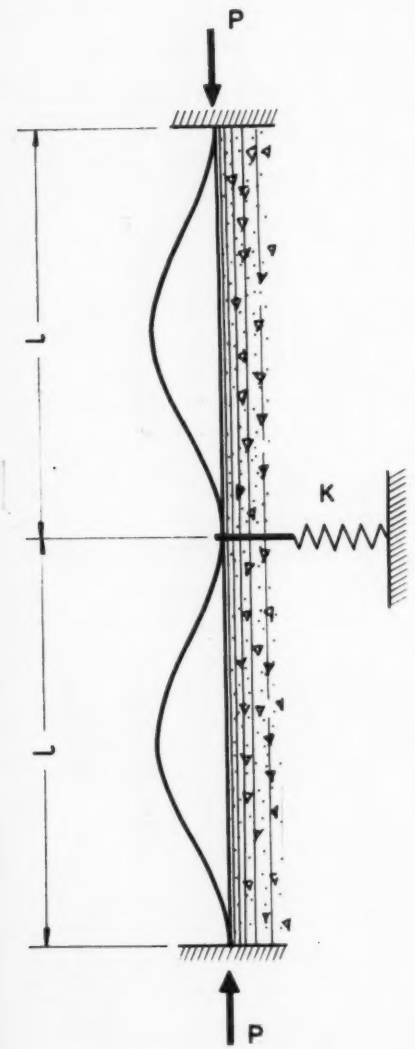


Fig. 3—Idealized mechanism of longitudinal bar buckling

Notation

A' = effective cross-sectional area of tie wire or bar

A_t = actual cross-sectional area of tie wire or bar

B = numerical coefficient

C = longitudinal bar end restraint coefficient

D = diameter of the longitudinal reinforcing bar

E = modulus of elasticity

E' = effective modulus of elasticity of tie wire or bar

E_t = tangent modulus of elasticity

I = moment of inertia of longtudinal bar cross section

I' = moment of inertia of tie bar cross section

P = axial load in the longitudinal reinforcing bar; critical buckling load

U = potential energy

a_1, a_2 = maximum amplitudes of buckling modes

b = core dimension

b' = effective length of tie wire or bar

d = diameter of tie wire or bar

f = unit stress

f_{cr} = critical buckling stress

f_y = yield stress

k = spring constant of lateral tie wire or bar

l = tie spacing

m = numerical coefficient

r = radius of gyration of longitudinal bar

x, y = coordinates

TIE SPACING AS GOVERNED BY BUCKLING OF LONGITUDINAL REINFORCEMENT

With the loss of the concrete cover either due to imposed loads or due to accidental damage, the longitudinal steel reinforcement may begin to yield and buckle between the lateral ties. The critical buckling stress f_{cr} of the longitudinal bars is determined by the bar diameter D, by the mechanical properties of the reinforcing steel, by the tie spacing l, and by the mode of buckling, Fig. 3. To obtain the maximum effectiveness of the longitudinal reinforcement, lateral ties should be spaced so as to permit development of critical buckling stress equal to the yield point stress, even in the absence of restraint provided by the outer concrete cover.

The mode of buckling of the longitudinal bars depends partly on the extent of spalling of the outer shell and partly on the arrangement and size of lateral ties. Lateral displacement of the bar at tie level would reduce appreciably the magnitude of the critical buckling stress and thus would reduce the load carrying capacity of the column. Therefore, it is desirable to provide an arrangement of ties which will prevent lateral displacement of each bar at the level of the tie. It is assumed here that the lateral ties are sufficiently rigid and that the lateral displacements of longitudinal bars at the tie level are negligible.

As the ratio of the spacing l to bar diameter D varies, either elastic or plastic buckling may govern the capacity of the bar, and the exact analysis becomes somewhat complex. A simplifying idealization is proposed here to establish a rational criterion for selection of tie spacing. It is assumed that the critical buckling stress f_{cr} can be defined as

$$f_{cr} = C\pi^2 E_t \left(\frac{l}{r}\right)^2 \qquad (1)$$

where C is the end restraint coefficient, E_t is the tangent modulus corresponding to stress f_{cr}, l is the unsupported length and r is the radius of gyration of the reinforcing bar.

To obtain maximum effectiveness of the longitudinal reinforcement, lateral ties should be spaced at a distance l which will permit development of yield point stress f_y just before buckling. Using the conditions $f_{cr} = f_y$, the ratio l/r is found from Eq. (1):

$$\frac{l}{r} = \left(\frac{C\pi^2 E_t}{f_y}\right)^{1/2} \qquad (2)$$

As $r = D/4$, where D is the diameter of the longitudinal bar, Eq. (2) may be written as follows:

$$\frac{l}{D} = B\left(\frac{E_t}{f_y}\right)^{1/2} \qquad (3)$$

where $B = \frac{1}{2}\pi C^{1/2}$.

The value E_t and f_y should be taken from the stress-strain diagram of the particular steel reinforcing bar used. For example, for the intermediate grade steel bar used by Hognestad in his column study,[7] the values of f_y and E_t were 43.6 and 10,000 kips per sq in., respectively. Using value of $C = 2$, the limiting value of l/D from Eq. (3) would be 17.

It is seen from Eq. (3) that a rational l/D ratio cannot be specified as a constant, but must depend on the values of E_t and f_y for the various types of steel used as reinforcement. With increased use of high strength steels, particularly for column reinforcement, determination of a suitable tie spacing becomes an important problem. For example, assuming $E_t = 10{,}000$ kips per sq in. and $C = 2$, and varying values of f_y, the l/D ratios listed in Table 1 are obtained from Eq. (3).

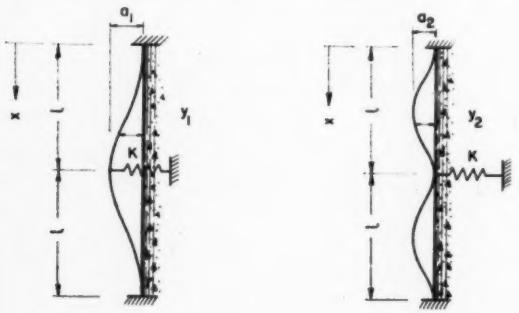
TABLE 1 — TABULATION OF l/D RATIOS OBTAINED FROM EQ. (3)

f_y, ksi	33	45	60	75	100
l/D	19.5	16.5	14.5	13	11

TIE SIZE AS GOVERNED BY LONGITUDINAL BUCKLING

To prevent the longitudinal bar from premature buckling due to inadequate stiffness of the ties, the ties must have a certain minimum diameter. The size of the ties can be determined approximately by solving the following idealized problem.

Consider a segment of a reinforcing bar having a length $2l$, Fig. 4, clamped at the ends, with an intermediate elastic "spring" support at midlength, with the concrete shell of the column spalled off and a continuous support along one side which prevents inward buckling. Solution of the problem requires determination of the smallest value of spring constant k of the intermediate support which would prevent buckling by lateral displacement at midlength. This problem can be solved approximately by the Ritz method.[8] The deflected shape of the



First Possible Mode of Buckling (a)

Second Possible Mode of Buckling (b)

Fig. 4 — Idealized buckling shapes

buckled bar may be approximated by the sum of two functions, defined by the first two possible buckling modes, Fig. 4 as follows:

$$y = y_1 + y_2 = \frac{a_1}{2}\left(1 - \cos\frac{\pi x}{l}\right) + \frac{a_2}{2}\left(1 - \cos\frac{2\pi x}{l}\right) \quad (4)$$

The total potential energy U of the system including the energy stored in the elastic spring and the energy due to the shortening of the column can be expressed as a function of a_1, a_2, k, P, l, and EI, the flexural rigidity of the longitudinal reinforcement, as follows:

$$U = \int_0^{2l} \frac{EI}{2}\left(\frac{d^2y}{dx^2}\right)^2 dx + \tfrac{1}{2}ka_1^2 - \frac{P}{2}\int_0^{2l}\left(\frac{dy}{dx}\right)^2 dx \quad (5)$$

where moment of inertia of longitudinal bar is $I = (\pi D^4/64)$. By completing the operations indicated in Eq. (5) and taking advantage of the orthogonality of the deflection modes, the following equation is obtained.

$$U = \frac{EI\pi^4 a_1^2}{8l^3} + \frac{2EI\pi^4 a_2^2}{l^3} + \tfrac{1}{2}ka_1^2 - \frac{Pa_1^2\pi^2}{8l} - \frac{Pa_2^2\pi^2}{2l} \quad (6)$$

By minimizing the energy with respect to each of the maximum amplitudes, the buckling load, P, and the required spring stiffness, k, are obtained as follows:

$$\frac{\partial U}{\partial a_1} = 0 = \frac{EI\pi^4 a_1}{4l^3} + ka_1 - \frac{P\pi^2 a_1}{4l} \quad (7a)$$

$$\frac{\partial U}{\partial a_2} = 0 = \frac{4EI\pi^4 a_2}{l^3} - \frac{P\pi^2 a_2}{l} \quad (7b)$$

Solving for P and k:

$$P = \frac{4\pi^2 EI}{l^2} \quad (8a)$$

$$k = \frac{3\pi^4 EI}{4l^3} \quad (8b)$$

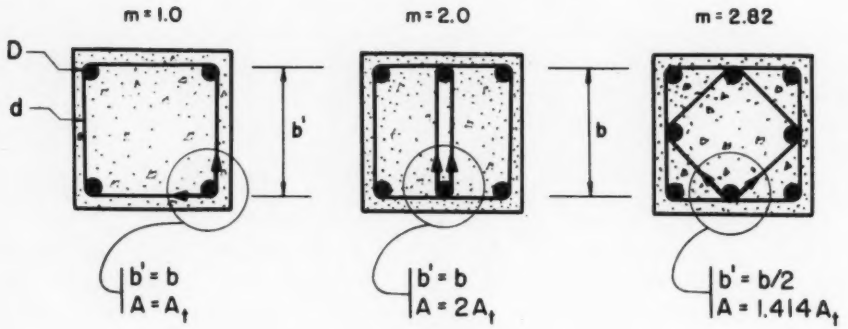
Two types of lateral support may be considered. In one, all of the longitudinal bars are supported laterally by direct tensile constraint of a tie, as in Fig. 5. Assuming that the tie acts as an equivalent elastic rod the stiffness is defined by $k = A'E'/b'$, where A' is the effective cross-sectional area of the tie, b' is the effective length of the tie, and E' is the effective modulus of the tie. Equating the two expressions for k,

$$k = \frac{3\pi^4 EI}{4l^3} = \frac{A'E'}{b'} \quad (9)$$

the effective cross-sectional area A' of the tie is,

$$A' = \frac{3\pi^4 E b' I}{4E' l^3} \quad (10)$$

$$m = \frac{A'}{A_t} \cdot \frac{b}{b'}$$



$$\frac{d}{D} = \sqrt{\frac{4.56}{m} \cdot \frac{b'}{l} \cdot \frac{D}{l}}$$

Fig. 5—Lateral tie effective length and area

For different arrangements of ties, the effective area A' and the effective length b' may be related to tie-bar cross-sectional area A_t and core dimension b as follows:

$$\frac{A'}{b'} = m\left(\frac{A_t}{b}\right) \quad \text{(11)}$$

Substituting Eq. (11) into Eq. (10), using $I = \pi D^4/64$, and assuming $E' = E$, the following relation is obtained:

$$\frac{d}{D} = \frac{D}{l}\left(\frac{4.56\,b}{ml}\right)^{1/2} \quad \text{(12)}$$

For common arrangements of lateral ties, the values of b', A', and m are shown in Fig. 5. For the case when $A' = A_t$, $b' = b$, and $b = l = 15D$, the required diameter d of the tie is determined from Eq. (12) to be $d = 0.142D$. Thus for a 1¾ in. diameter longitudinal bar, only a ¼ in. diameter tie would be required. Proportionally smaller ties would be required for longitudinal bars of smaller diameter.

Another type of lateral support may be considered when the longitudinal bar is constrained by the flexural stiffness of a tie. For a typical case shown in Fig. 6, the deformation of the tie may be considered approximately equivalent to that of a fixed-ended beam loaded by a concentrated load W at midspan, so that the spring constant

$$k = \frac{W}{\Delta} = \frac{192\,E'\,I'}{b^3}$$

where I' is the moment of inertia of tie cross section. Equating the two expressions for k,

$$\frac{3\pi^4 EI}{4l^3} = \frac{192 E' I'}{b^3} \quad \text{(13)}$$

and with $E = E'$, and $A' = A_t$:

$$\frac{d}{D} = 0.785\left(\frac{b}{l}\right)^{3/4} \quad \text{(14)}$$

When the value of b/l is of the order of magnitude of unity, the use of ties in flexure for effective lateral tie support becomes prohibitive, since the tie diameter must be approximately ¾ that of the longitudinal bar. For small values of b/l, say ⅓, use of ties in flexure may be practically feasible, as the tie diameter must be approximately ⅓ that of the longitudinal bar.

EXPERIMENTAL STUDY OF TIE SIZE REQUIREMENTS

The tie requirements for reinforced concrete columns developed in the preceding section indicate that the size of ties may be reduced materially from that used in current design practice. To verify partially the validity of this hypothesis, four columns were tested in the Engineering Materials Laboratory at the University of California, Berkeley. The variables in this pilot test program were the size and arrangement of ties, the amount and arrangement of longitudinal reinforcement, and the concrete strength.

The columns were 8 x 8 x 60 in., reinforced with #5 deformed bars of intermediate grade billet steel with a yield stress of 44.5 kips per sq in. and an ultimate tensile strength of 77.5 kips per sq in. Fig. 7

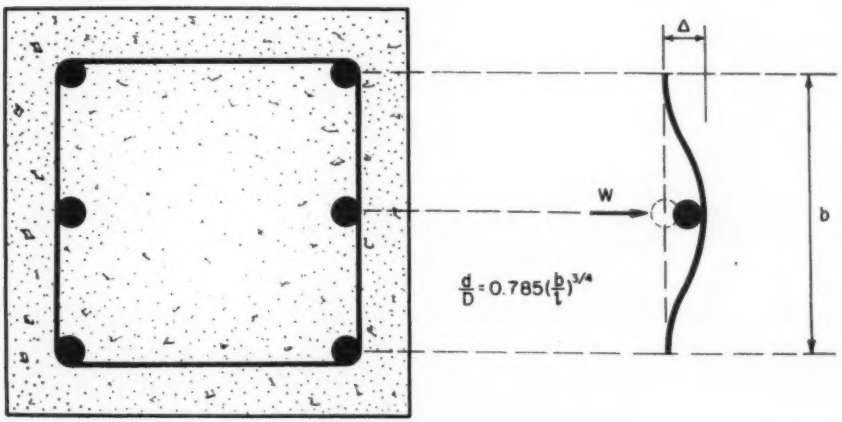


Fig. 6—Lateral tie flexural restraint

shows the details of the specimen; #2 deformed bars with a yield stress of 52.1 kips per sq in. and 1/16 in. diameter steel wire with a yield stress of 48.2 kips per sq in. were used as ties. The columns were cast vertically, two at a time, A-6 and S-6 in one placement and A-8 and S-8 in a separate placement. The six-bar columns were cured under wet burlap for 14 days, and air dried for 45 days; the average compressive strength of 6 x 12 in. cylinders cured under the same conditions as the columns was 6 kips per sq in. The eight-bar columns

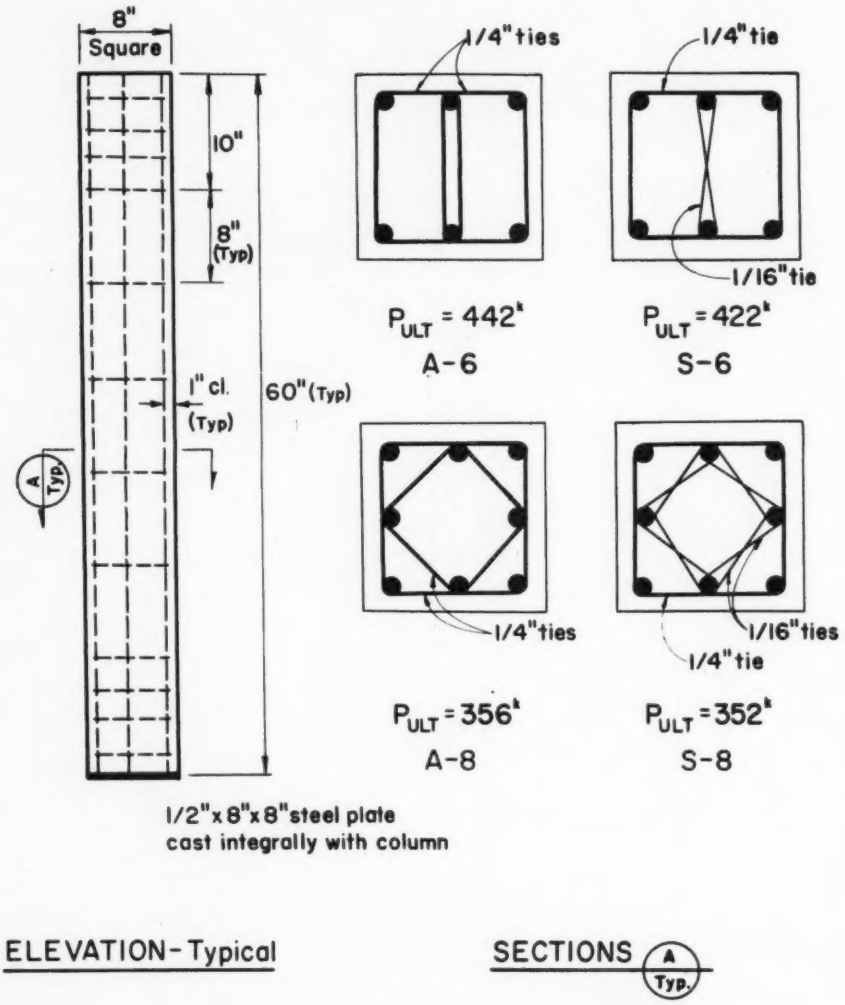


Fig. 7—Details of column test specimen

were cured under wet burlap for 10 days and air dried for 18 days; the average compressive strength of the cylinders cured under the same conditions was 4 kips per sq in.

The columns were subjected to axial compression in a 4,000,000 lb testing machine; the loads were applied in 20 kip increments and strain measurements of axial shortening were taken at each increment of

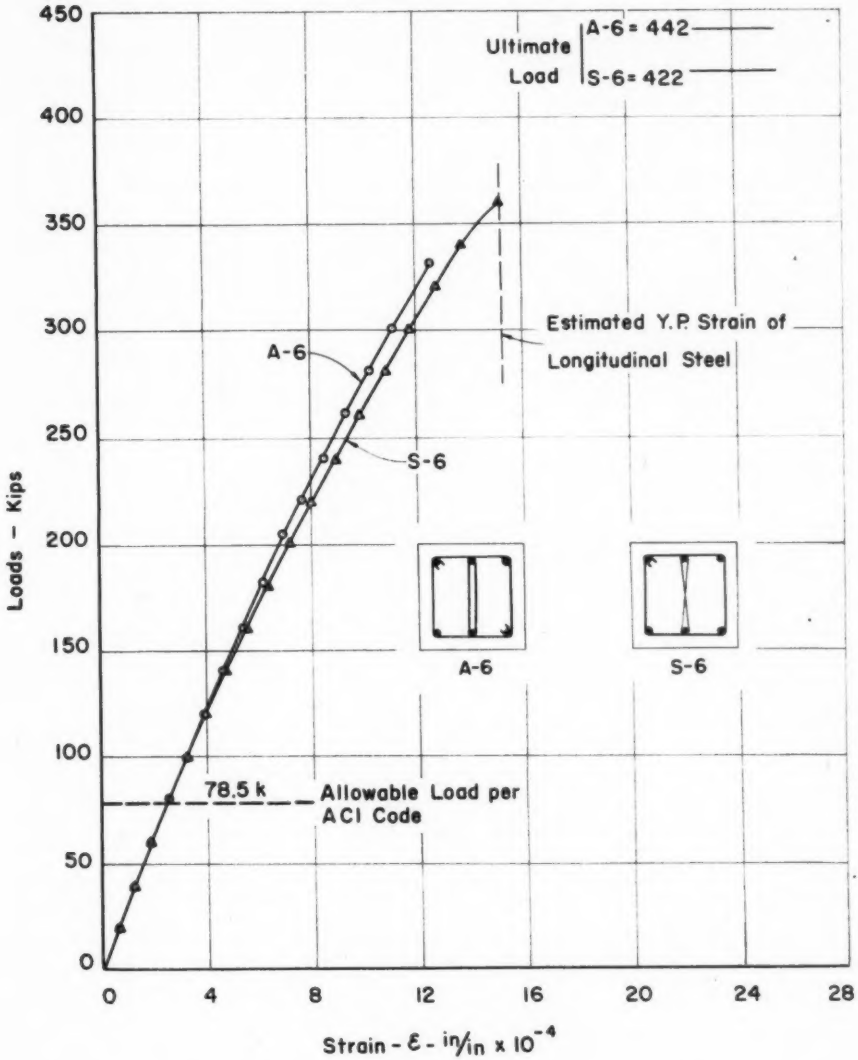


Fig. 8—Load-strain curves for Columns A-6 and S-6

load. A compressometer, having a gage length of 39 in., was used on each column. Compressometer readings were taken up to a unit strain of 15×10^{-4} in. per in. (just below the yield point of the longitudinal steel) at which time the load was maintained while the compressometer was removed and then the loading was continued on to failure of the column. The load-strain diagrams are shown in Fig. 8 and 9, and the test results are summarized in Table 2. Within the range of measured deflections, the load-deflection relationship was nearly linear, with some slight increase in strain at the upper values of column load. No noticeable cracks appeared until the ultimate load value was reached.

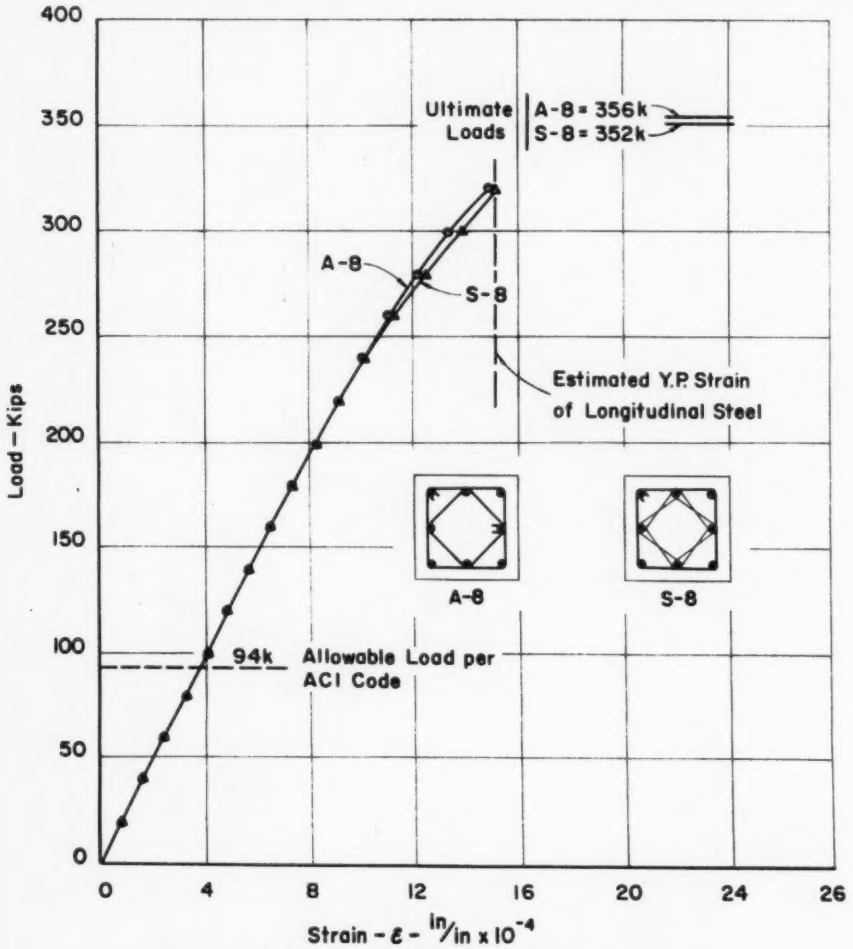


Fig. 9—Load-strain curves for Columns A-8 and S-8

TABLE 2 — SUMMARY OF TEST RESULTS

Column designation	Avg f_c', psi	Longitudinal steel		Ties			Ultimate load, kips
		No. and size of bars	f_y, ksi	Spacing, in.	No. and size of ties	f_y, ksi	
A-6	6	6 #5	44.5	8	2 #2	52.1	442
S-6	6	6 #5	44.5	8	1 #2 1 wire, 1/16 in. dia	52.1 48.2	422
A-8	4	8 #5	44.5	8	2 #2	52.1	356
S-8	4	8 #5	44.5	8	1 #2 1 wire, 1/16 in. dia	52.1 48.2	352

At this point the column was yielding slowly, but was able to maintain full ultimate load until light cracking in the shell was observed; then the load began to drop, the shell spalled, the steel reinforcement buckled outward between the ties, and the core failed in a characteristic shear-cone, Fig. 10.

Comparison of load-deflection characteristics and of ultimate loads for the companion specimen with six and eight bars indicates no significant difference between the columns with conventional ties and

Fig. 10—Typical failure of test specimen

those with light interior ties. The small differences in the ultimate loads are considered to be within the range of experimental variation.

In all columns the conventional ¼ in. diameter ties separated at the corner lap splices. This was believed to have occurred as a secondary consequence of failure; no apparent yielding of these ties had taken place. In the S-6 and S-8 columns the 1/16 in. diameter ties had yielded and some had ruptured; this also was believed to be a secondary consequence of failure, as the ultimate load was not materially affected.

DISCUSSION

Analysis of idealized behavior of reinforced concrete tied columns resulted in development of the following criteria for design of lateral ties:

1. To develop maximum strength of concrete core, tie spacing should not exceed a distance approximately equal to the least lateral dimension of the column.

2. To prevent buckling of the longitudinal reinforcement at a stress below yield, tie spacing should not exceed a distance l defined by Eq. (3), $l = BD(E_t/f_y)^{½}$.

3. To prevent reduction of the buckling strength of the longitudinal reinforcement by excessive deformation of the ties, the ratio of tie diameter to bar diameter is defined by geometry of tie arrangement, by tie spacing, and by diameter of longitudinal bar. For special cases Eq. (12) and Eq. (14) state possible design criteria.

The pilot tests described above were preliminary; nevertheless, they indicated that the size of some interior ties may be reduced materially without reducing the load carrying capacity of the columns. In specimens S-6 and S-8 use of wire ties effected an 8 percent saving in the weight of steel reinforcement as compared with the weight of steel in Specimens A-6 and A-8. Also the light ties eliminated some of the congestion of reinforcement in the core of the column and thus resulted in greater ease of placing concrete.

The design criteria proposed above suggests that lighter ties than currently used would be adequate for reinforced concrete columns, but that in columns with high strength longitudinal reinforcing bars closer tie spacing than presently required should be used. Realizing the idealization made in this study and the exploratory nature and the limited scope of the testing program described in this paper, the authors consider further experimental verification highly desirable.

ACKNOWLEDGMENTS

The authors gratefully acknowledge valuable suggestions received from R. A. Westmann, and the general assistance of the laboratory staff of the Division of Structural Engineering and Structural Mechanics, Department of Civil Engineering, at the University of California, Berkeley.

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Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Estipulaciones para Tirantes de Columnas de Hormigón Armado

Se describe la mecánica del fallo de columnas de hormigón armado con tirantes laterales. También se deduce una norma racional para espaciar y determinar el tamaño de los tirantes laterales y se describen pruebas índices que verifican la misma.

Les Exigences de Tirants pour Colonnes en Béton Armé

Décrit le mécanisme de rupture de colonne en béton armé à tirants latéraux. On en déduit de critères rationnels de l'espacement et des dimensions de tirants latéraux, et on décrit des essais témoins vérifiant ces critères.

Anforderungen an die Verankerung bei Eisenbetonsäulen

Der Vorgang beim Versagen von Eisenbetonsäulen mit seitlichen Verankerungen wird beschrieben. Rationelle Kriterien für die Verteilung und Grösse seitlicher Verankerungen werden abgeleitet und Vorversuche, die diese Kriterien beweisen, werden beschrieben.

Title No. 58-27

Reinforced Concrete Failures During Earthquakes

By ROGER DIAZ DE COSSIO and EMILIO ROSENBLUETH

Photographic evidence is presented and discussed of different types of failures occurring to reinforced concrete members during earthquakes. For the most part, examples are shown from three recent Mexican earthquakes: the 1956 Oaxaca, the 1957 Mexico City, and the 1959 Coatzacoalcos-Jaltipan. Some examples are also shown from the 1923 Kanto and 1948 Fukui Japanese earthquakes, and from the recent 1960 Chilean earthquakes. The major types of failures observed were shear and diagonal tension, beam or slab-column connection failures, excessive bending, and tension. Most of the damage observed was due to poor construction practices and oversimplifications in the design. However, a study of the photographic evidence makes designers and construction men aware of the major types of damage to guard against. Also, types of failure observed again and again, in different structures and in different regions, point out weaknesses in local building codes.

■ TYPICAL EXAMPLES ARE GIVEN in this paper of the most common types of failures in reinforced concrete members occurring during earthquakes. Photographs are shown from the 1923 Kanto and 1948 Fukui Japanese earthquakes; a few from the 1960 Chilean earthquakes; and for the most part, from three recent Mexican earthquakes: the 1956 Oaxaca, the 1957 Mexico City, and the 1959 Coatzacoalcos-Jáltipan.

It is realized that most failures were due to poor construction practice and/or lack of consideration of seismic forces, or oversimplifications in the design. It is usually difficult, from photographic evidence alone, to determine the primary cause of failure. However, a study of this material is of value in that it points out major structural damages to guard against. It makes designers and construction men aware of the important details to consider in the design and construction of reinforced concrete structures in seismic regions. Also, a constantly repeated type of failure emphasizes weaknesses in local building codes.

The photographs are grouped according to the type of failure or damage observed: (1) shear and diagonal tension; (2) beam or slab-column connections; (3) bending, reversal of bending moments, and tension; (4) splices; and (5) whole structures deserving a special discussion.

SHEAR AND DIAGONAL TENSION FAILURES

By far, the most common type of failure observed during recent Mexican earthquakes was shear and diagonal tension. This type of failure was also frequently observed during the 1923 Kanto earthquake. Shear failures appeared in both beams and columns. Fig. 1-3 are photographs of inclined cracking and shear failures in beams, and Fig. 4-7 are photographs of shear failures in columns. In the beams depicted in Fig. 2b and 2c, it is observed that the major inclined cracks tend to go through sections where the steel was cut off.

Also important is the fact that shear failures were almost as common in columns as in beams. This shows that much more attention should be given to the design of web reinforcement in columns especially, since most building codes require ties or web reinforcement in columns by specifying only maximum spacing.

Spirally reinforced columns are not so frequently used as tied columns, but the fact remains that no spiral column failures were observed. This again points out the fundamental weakness in present specifications for tied columns.

Fig. 1—Three inclined cracks in a beam

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ACI member **Roger Diaz de Cossio** is the investigator in charge of research on concrete and masonry structures at the Institute of Engineering, University of Mexico. He is a member of ACI-ASCE Committee 341, Reinforced Concrete Columns, and ACI Committee 339, Allowable Stresses in Reinforcement.

The designer of columns is seldom aware of the need of considering such a member as one subjected to large amounts of bending and shear, in addition to axial load. It is the authors' opinion that differentiation between beams and columns is artificial; such practice leads to dangerous misconceptions of basic behavior of the members. A more rational approach is to consider flexural members subjected to axial load, bending, and shear, independently of whether they are vertically or horizontally oriented.

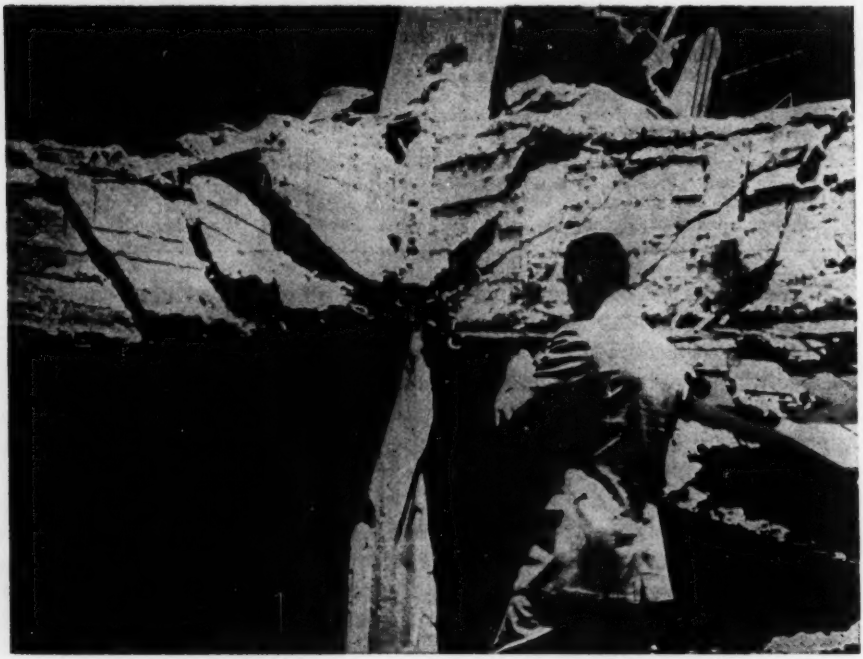
An interesting correlation exists between building code requirements and earthquake damage. In recent Mexican earthquakes a good portion of the damage was due to shear and diagonal tension, while in the 1960 Chilean earthquakes few shear failures were observed. This is explained by the fact that Mexico follows essentially American practice (ACI 318-51), while Chile follows German practice (DIN). Especially with regard to maximum spacing of stirrups in beams, Chilean requirements are slightly more severe than the original DIN specifications. The 1958 Chilean specification[1] for web reinforcement reads as follows:

Article 30

1. To absorb diagonal tension in slabs, beams and frames, inclined bars shall preferably be used . . .
2. The maximum shearing stress in stirrups shall not exceed the values in Table II increased by one third.*
3. In beams having a span to depth ratio smaller than two, inclined stresses shall be taken by inclined bars and/or mesh made of vertical and horizontal reinforcement . . .
4. In beams of width greater than 18 in., stirrups of more than two vertical branches shall be used.
5. Spacing of stirrups shall not exceed 12 in., nor the width of the web, nor half the beam depth.
6. Open stirrups are allowed only in simply supported beams, and only if they are properly anchored in compression zones. No open stirrups are allowed in doubly reinforced beams.
7. The horizontal distance between inclined bars shall be not more than twice the depth of the beam.

As can be seen, the requirement under No. 7 concerning spacing of inclined bars is too liberal. However, most of the beams where failure

*The values in Table II range between 56 and 140 psi, depending on the type of concrete used, and are slightly larger for slabs.



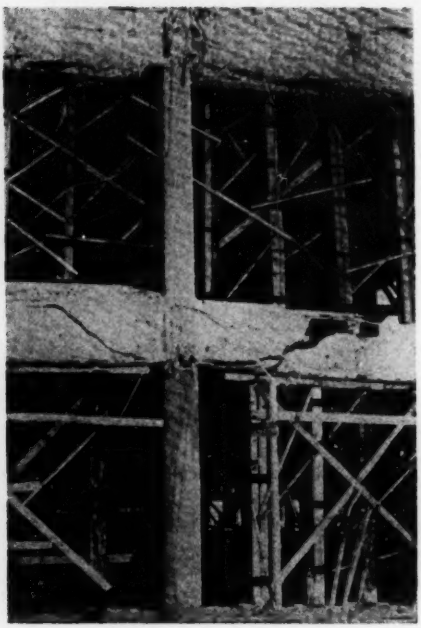


Fig. 2a—(top) Shear failure of beams in a school building

Fig. 2b—(left) Shear failure showing how cracks go through cut off points

Fig. 2c—(bottom) Shear failure

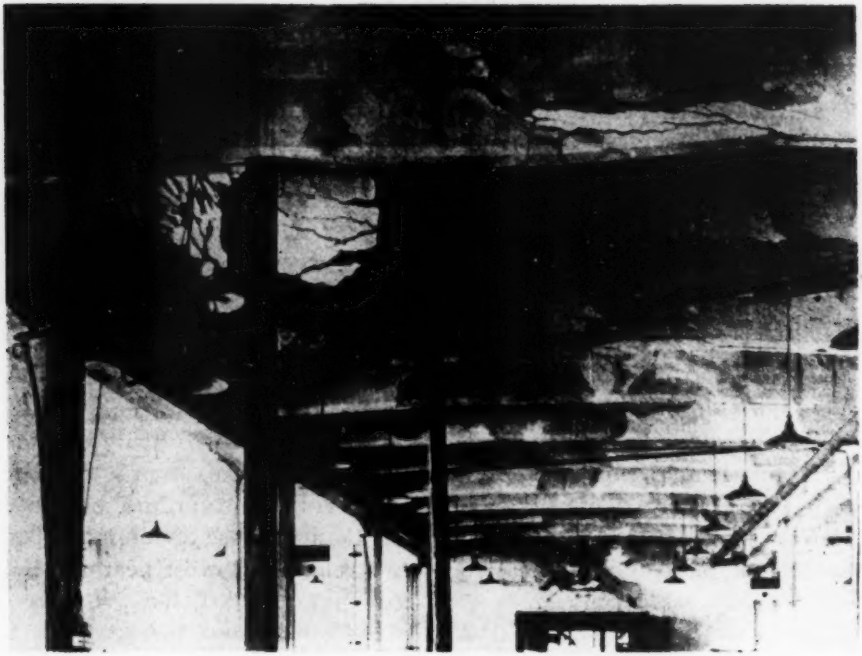
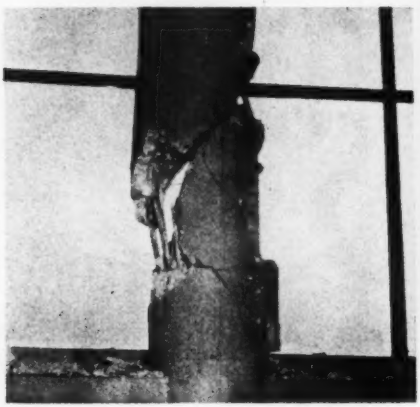


Fig. 3—(top) Shear-splitting failure

Fig. 4—(bottom) Circular column failure

Fig. 5—(right) Typical column shear failure showing a striking similarity to laboratory beam specimens



was noted were also reinforced with stirrups, and the spacing of this reinforcement is tightly controlled. This may explain the scarcity of shear failures during the last Chilean earthquakes.

As mentioned before, shear failures were frequent during the 1923 Kanto earthquake in Japan. It should be noted that at the time there were no building regulations for the city of Tokyo. Fig. 3 shows a shear-splitting failure which occurred in an industrial building during the Kanto earthquake.

In Fig. 5 and 7, shear failures in columns of circular cross section are shown. Knowledge of shear strength of members of circular cross sections is scant indeed. The problem deserves experimental investigation, since many of the columns which failed in shear in Mexico were of circular cross section. Shear failures were also frequent in regions near the ends of a column, as shown in Fig. 6 and 7. In fact, it was sometimes difficult to determine whether failure was truly due to shear and bending, or to crushing of the end concrete, as in a cylinder tested in compression. Notably, the latter type of failure occurred almost systematically near the top rather than the base of columns. The situation reflects the fact that concrete strength is lowest near the top of vertically cast members.

CONNECTIONS OF COLUMNS TO BEAMS AND SLABS

The second most frequent source of damage in reinforced concrete structures during earthquakes is due to poor connections between beams and columns, and between columns and slabs. Fig. 8-13 show failures of this type occurring in Japan, Chile, and Mexico. One common manner of failure was as shown in Fig. 8. Due to the horizontal

Fig. 6—Column shear failure near beam connection

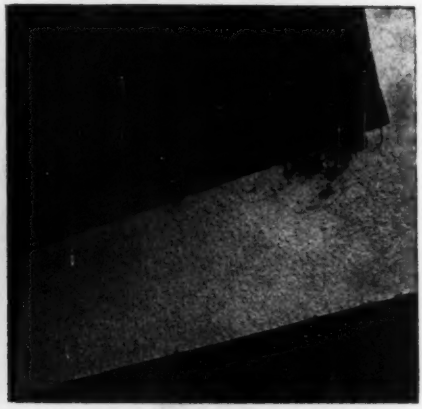


Fig. 7—Shear failure of circular column

movements of beams and slab, the top column concrete crushes, the reinforcing bars, unable to take the weight, buckle, and floor level drops considerably. Combined with this phenomenon, are the effects of long time redistribution of stresses between column steel and concrete, which tend to facilitate buckling of column vertical bars, since steel stresses may reach levels considerably in excess of their normal working values for vertical load. Often this type of failure can be correlated with absence of stirrups (web reinforcement) at the intersection of beams and columns. It is common practice to stop ties near the end of the column free span. If the concrete at the connection is crushed, vertical bars can easily buckle out. Also, if the ties near the ends are not closely spaced, vertical reinforcing bars will buckle out between them, as shown in Fig. 10.

Column compression failures in an internal row of a flat slab building are shown in Fig. 9. It is to be noted that failures occurred immediately beneath the column capitals.

A different type of failure is observed at corner beam-column connections (Fig. 11-13). In this case the concrete is not restrained horizontally and it spalls. Sometimes, as shown in Fig. 11, failure at the connection is combined with a local shear failure.

Fig. 8—Beam-column connection failure with dropping of roof

Fig. 9—Compression and shear failures of columns in a flat slab building

Fig. 14—Failure of column by bending

Fig. 15 — Compression failure at the bottom of a column

Fig. 16 — Typical column compression failure

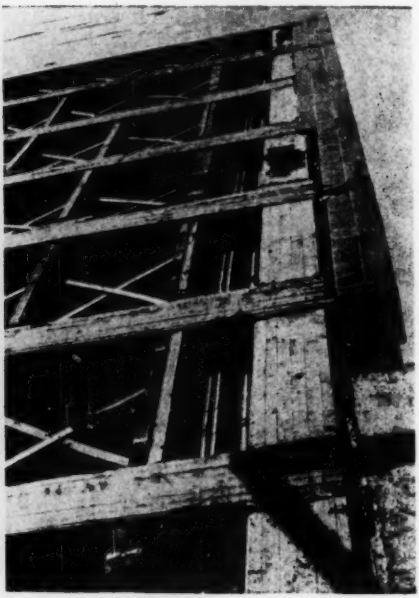


Fig. 17—Damage by reversal of bending moments

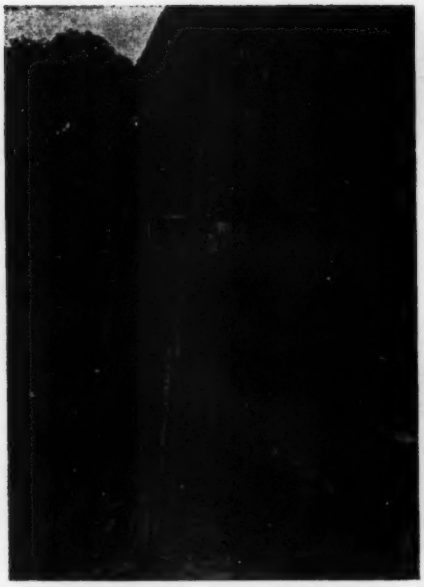


Fig. 18—Moment failure of posts, some of which appear to have large tension cracks

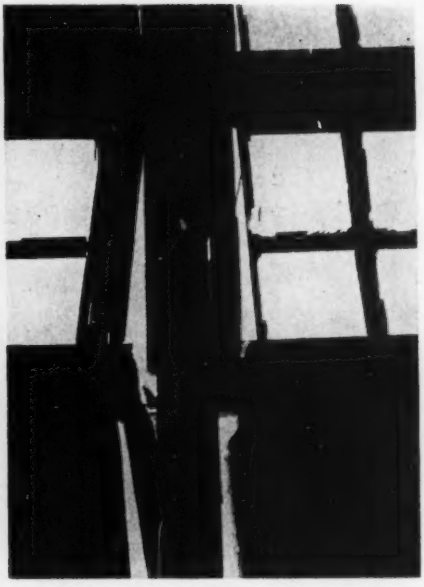


Fig. 19—Tension failures of secondary girders due to large horizontal movements

Fig. 20—Column failure by insufficient steel and too much cover

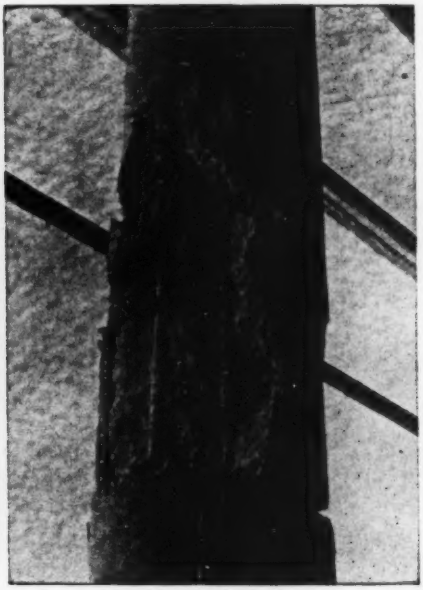


Fig. 21—Tension side of circular column failed by bending

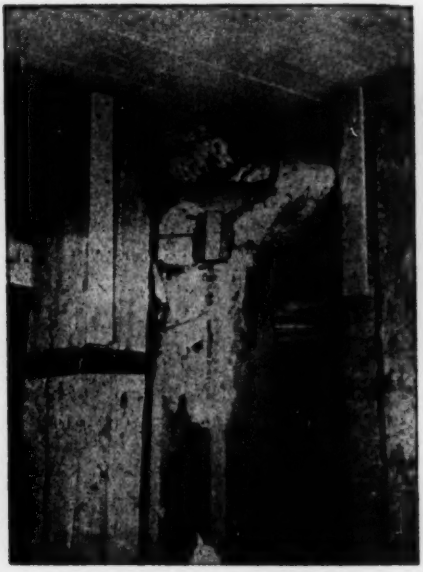


Fig. 10—Buckling out of vertical reinforcing steel

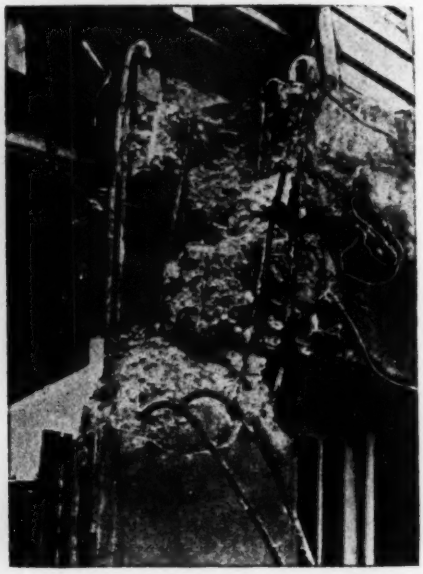


Fig. 11—Beam-column connection failure at end beam due to insufficient anchorage

Fig. 12—Typical appearance of failed end beam-column connection

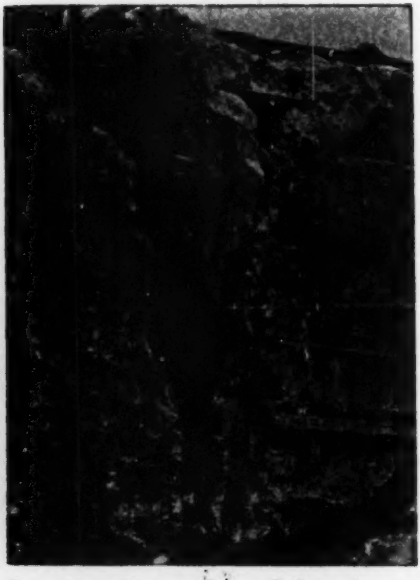


Fig. 13—Beam-column failure by insufficient cement or air voids

BENDING, REVERSAL OF BENDING MOMENTS, AND TENSION

Some failures in bending have been observed during recent Mexican earthquakes. A few examples are shown in Fig. 14-16. Cases of combined bending and compression failures are shown in Fig. 15 and 16, the steel buckling out between ties. The large spacing between ties (which, nevertheless, sometimes satisfy code requirements) should be noted; this was typical in most of the damaged columns.

Damage due to reversal of bending moments not properly considered in design, is shown in Fig. 17. The concrete spalled at the bottom of the beams near the columns.

Fig. 18 shows damage caused by distortion of the whole structure, inducing tension and bending moments in the posts. In Fig. 19, secondary members clearly failed by axial tension, caused by lateral movements of the frame.

The failures discussed in this section were due mostly to careless design or construction practices. Good practice will in general produce structures in which these types of failure are not likely to happen.

SPLICES AND MISPLACEMENT OF REINFORCEMENT

Failures due to misplacing of reinforcement or localized at splices, usually fall into one of the types described above or are combined therewith. Fig. 20-23 depict some of these failures. The column in Fig. 20 failed due to improper placing of longitudinal reinforcement; longitudinal bars were quite deep into the column.

With reference to Fig. 23, from the recent Chilean earthquakes, it should be noted that thin stirrups were closely spaced, yet did not prove effective in preventing failure.

MISCELLANEOUS STRUCTURES

Fig. 24a and 24b show photographs of a two-story bandstand that had been cast a few days prior to the earthquake. The columns collapsed, and the whole structure went down vertically, formwork and all. The lower end of one of the base columns can be seen in Fig. 24b.

Photographs of a collapsed school building in Mexico appear in Fig. 25a-25f. One complete row of external columns failed and several different types of damage occurred in inside members. This structure summarizes, in fact, most of the types of failures discussed earlier. Indeed, Fig. 25b and 25c again show the crushing of concrete at the top of columns and the buckling of vertical reinforcing bars. Shear splitting failures of interior beams are shown in Fig. 25c. Failure at a beam-column connection is shown in Fig. 25d, and a shear failure of one of the lower columns is shown in Fig. 25e.

Failures of two water tanks, one in Japan during the Kanto earthquake of 1923, the other in Chile during the earthquake of 1960, are shown in Fig. 26 and 27, respectively. It is interesting to note the similarity between the type of structure and the type of failure in the two cases. Both tanks had a large concrete mass at the top, supported by rectangular columns and haunched stiffening beams. Two facts should be observed: the main cracks in the beams follow the direction of the haunch (the reason for such selectivity is not clear), and the aspect of the failure suggests in both cases the action of large torsional moments in the beams.

CONCLUSIONS

From a study of photographic evidence, of which typical examples are presented in this paper, the following conclusions can be drawn with regard to design and construction practices in seismic regions:

1. Almost all the failures reported have been of the brittle type. Clearly, benefits are derived from a design in which ductile failures, with the accompanying redistribution of forces and formation of plastic hinges, practically eliminate the possibility of brittle fracture. This requires that the ductile types of failure—such as hinge formation in under-reinforced or heavily tied beams—take place at magnitudes of horizontal accelerations far smaller than those which would cause failure in shear of members with little such reinforcement, compression failure of tied columns, and the like.

2. It is of the utmost importance, in seismic regions, to give careful attention to the detailing of connections. All of the steel should be well anchored, and good practice calls for the presence of adequately spaced ties and stirrups where required to prevent buckling of longitudinal reinforcement, even at beam-column intersections and, in general, to provide good confining elements at and near the connections.

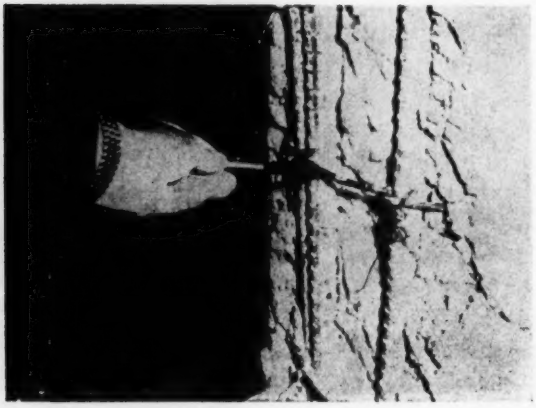
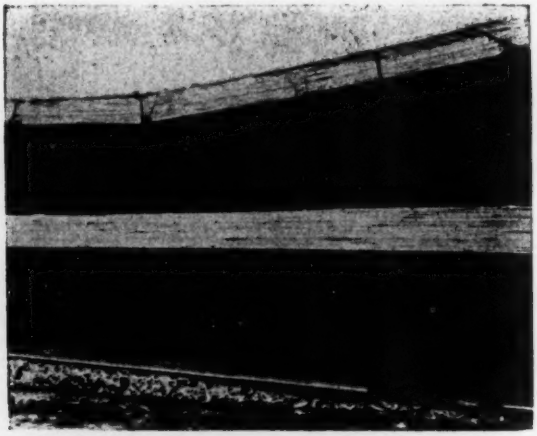


Fig. 22—Moment failure of tension side of circular column

Fig. 23 — Shear failure of column prompted by defective splice

Fig. 24a—General appearance of a plaza stand after failure

Fig. 24b—Close up of lower end of a base column shown in the foreground of Fig. 24a



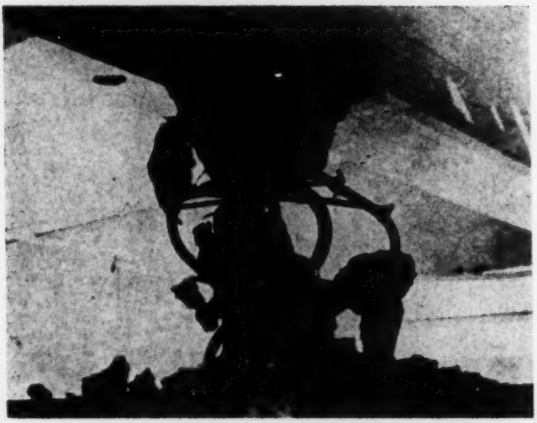
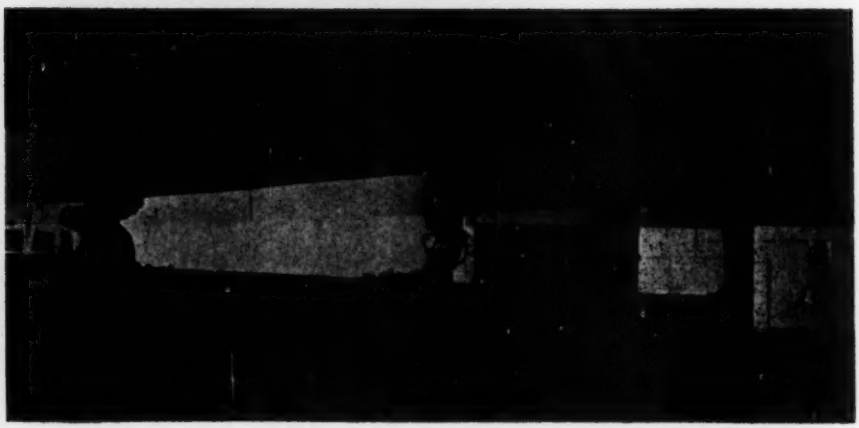


Fig. 25a—(top) Side view of a building partially collapsed

Fig. 25b — (left) Close-up of column

Fig. 25c — (bottom) Inside view of building showing crushing of concrete and subsequent buckling of column steel



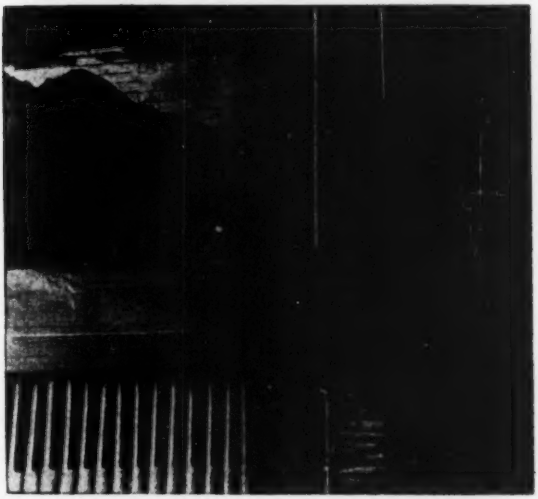
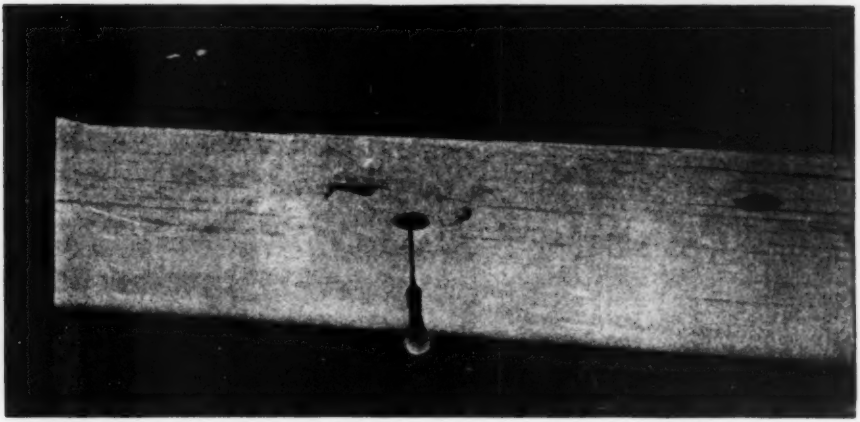


Fig. 25d—(top) Failure of beam-column connection

Fig. 25e—(right) Shear failure of first floor column

Fig. 25f — (bottom) Shear and splitting failures of roof beams



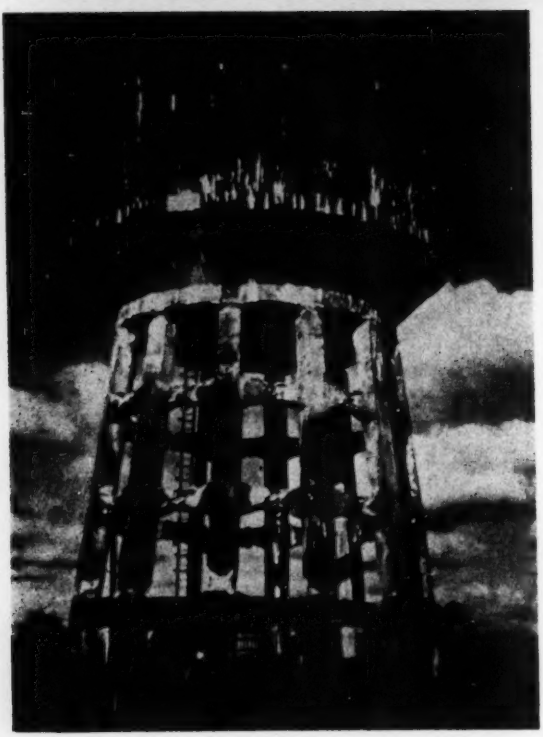
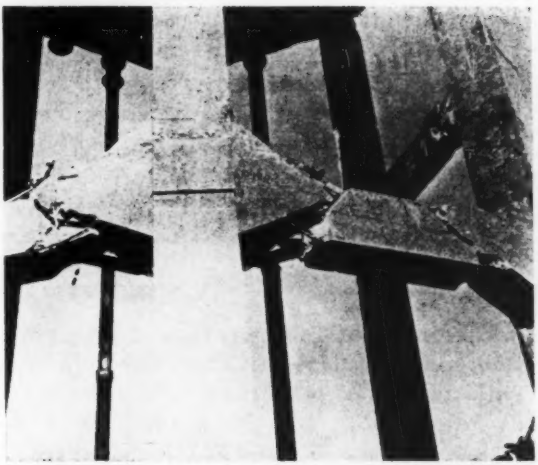


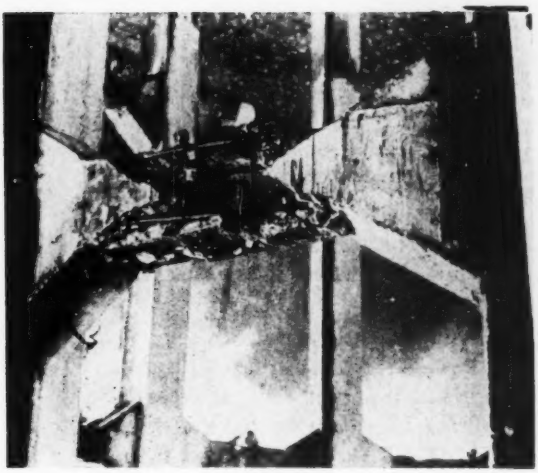
Fig. 26—(top) Failure of water tank showing inclined cracks following haunch lines

Fig. 27a—(bottom) Failure of water tank with badly cracked columns similar to those shown in Fig. 26

Fig. 27b — (top) General appearance of tank supports

Fig. 27c — (bottom) Detailed view of haunched beams





Also, concrete should be properly placed and vibrated, particularly at the connections, and the location of construction joints should be given careful consideration. All of this is advisable for any well built reinforced concrete structure, but is specially important for one built in a seismic region.

3. More attention should be given in building codes, especially in one as influential as ACI, to the spacing and confining action of column ties. In general, more experimental research is justified in this direction, chiefly in the range of close spacings. In this regard, one simple rule, that appears to be justified, is to space the ties closer together

near the ends of a column. In this way, both ductility and strength are raised, since concrete is weaker near the top end.

4. More information is needed with regard to shear strength of members subjected to axial load either tension or compression, bending, and shear. Information for members of circular cross section is nonexistent.

5. There is a need for greater knowledge of the rotation characteristics of connections, and of sections in the vicinity.

CLOSING REMARKS

The object of this paper has been to show actual field examples of reinforced concrete failures. The similarity of some of the examples with failures of laboratory specimens is sometimes striking, especially in shear.

It should be emphasized at this point that structures of other materials have also failed during earthquakes, and that, at least in Mexico City, the majority of structures are of reinforced concrete due to the local economy and most concrete structures withstand earthquakes successfully. During the 1957 earthquakes in Mexico City (4.5 million inhabitants), only three buildings collapsed, and they were clearly cases in which good construction and design practices were not followed.

The following comments refer specifically to the structures shown in the photographs in this paper and are made to avoid any possible misuse of this material for commercial purposes and point out in a general manner the main reason for which the structures were so severely damaged.

The structures shown in Fig. 3, 8, and 9 are from the 1923 Japanese earthquake. No building regulations existed in Tokyo at the time. Oversimplifications and lack of consideration of seismic forces in the design were the main causes for the damage to the structures shown in Fig. 4, 7, 14, 18-20, 24, and 25. The tanks in Fig. 26 and 27 constitute an undesirable type of structure for seismic regions.

In the remainder of the structures (Fig. 1, 2, 5, 6, 10-13, 15-17, and 21-23), the following causes can be mentioned in rapidly decreasing order of importance: defective construction techniques and inspection, oversimplifications in design and analysis, and design for a low seismic coefficient. Further, in some structures the damage may have been cumulative due to past earthquakes and other causes.

ACKNOWLEDGMENTS

The authors are grateful to Hajime Umemura and Rodrigo Flores, from the Universities of Tokyo and Chile, and to K. V. Steinbrugge, secretary, Earthquake Engineering Research Institute, for permission to reproduce the photographic material.

REFERENCE

1. "Official Building Regulations for the Republic of Chile," Inditecnor 63-4E, Reinforced Concrete, First Part, Revista Chilena de Racionalización, Chile, July 1957, p. 18. (In Spanish).

Presented at the 13th ACI regional meeting, Tuscon, Nov. 1, 1960. Title No. 58-27 is a part of copyrighted Journal of the American Concrete Institute, Proceedings V. 58, No. 5, Nov. 1961. Separate prints are available at 60 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Fallas en Concreto Reforzado Durante Sismos

En este trabajo se presenta y se discute evidencia fotográfica de los distintos tipos de fallas que ocurren en miembros de concreto reforzado durante sismos. La mayor parte de los casos mostrados son de tres sismos mexicancos recientes: el de Oaxaca en 1956, de la ciudad de México en 1957, y el de Coatzacoalcos-Jáltipan de 1959. Se muestran algunos ejemplos también de los temblores Japaneses, Kanto, 1923 y Fukui, 1948, y de los sismos Chilenos de 1960. Los principales tipos de falla que se observaron fueron en cortantes y tensión diagonal fallas en las juntas de vigas y losas con columnas, flexión excesiva, y tensión. La mayor parte del daño observado se debió a malas prácticas constructivas y sobresimplificaciones en el diseño. El estudio de la evidencia fotográfica recalca los principales tipos de daño. Los modos de falla que se observan repetidas veces, en distintas estructuras y en distintas regiones, señalan debilidades de los reglamentos vigentes.

Effondrements de Béton Armé Produits par Séisme

Présentation et discussion le l'évidence photographique de types variées d'effondrement de pièces en béton armé au cours de tremblements de terre. La plupart des exemples résulte de trois séismes récents en Mexique: à Oaxaca en 1956, à la ville de Mexique en 1957 et celui de 1959 à Coatzacialcos-Jaltipan. On présente également des examples résultants des séismes japonais: Nasuto, 1923 et Fukui, 1948; et du séisme récent (1960) en Chile. Les effondrements principaux qu'on a remarqués sont: traction due au éffort et traction diagonale, effondrement de réunion d'une poutre ou d'un colonne, la flexion ou la traction excessive. La plupart du dommage observé était dû à la construction défectueuse et la simplification exagérée de la structure. Une étude de ces photographies indiquera aux dessinateurs et aux ingénieurs les types principales de dommage qu'il faut prévenir. Aussi, les types d'effondrement observées maintes et maintes fois dans des structures variées et dans des régions différentes, indiquent des faiblesses des codes locaux, de la construction.

Eisenbetonbrüche bei Erdbeben

Fotografischer Beweis für verschiedene Arten des Versagens von Eisenbeton, die bei Erdbeben vorkommen, wird erbracht und diskutiert. Zum grössten Teil werden Beispiele von 3 kürzlich stattgefundenen mexikanischen Erdbeben gezeigt: Oaxaca 1956, Mexico City 1957 und Coatzacoalcos-Jaltipan 1959. Es werden ebenfalls einige Beispiele von den Erdbeben in Kanto 1923 und Fukui Japanese 1948 und von dem kürzlich im Jahre 1960 in Chile erfolgten Erdbeben gezeigt. Die hauptsächlich beobachteten Arten des Versagens waren Schub- und Diagonalsspannungsbrüche und Versagen der Verbindung zwischen Träger und Pfeiler oder Decke und Pfeiler, übermassige Durchbiegung und Spannung. Der grösste Teil des beobachteten Schadens war unzureichenden Arbeitsmethoden beim Bauen und zu grossen Vereinfachungen im Entwurf zuzuschreiben. Jedoch macht ein Studium der fotografischen Unterlagen Konstrukteure und Bauleute auf die Hauptschadensarten aufmerksam, gegen die man sich schützen muss. Ausserdem weisen die Arten des Versagens, die immer wieder in verschiedenen Strukturen und verschiedenen Bezirken beobachtet wurden, auf Schwachen in den örtlichen Bauvorschriften hin.

Title No. 58-28

Crack Propagation and the Fracture of Concrete

By M. F. KAPLAN

The Griffith crack theory of fracture strength is discussed. Tests were performed on concrete beams with crack-simulating notches, and two methods, which have been called the analytical and the direct experimental methods, were used to determine the critical strain-energy-release rate G_c associated with the rapid extension of the crack. There was good agreement between G_c values for beams with different notch depths and which were loaded both by the third-point and center-point methods. However, 3 x 4 x 16-in. beams gave somewhat larger G_c values than did 6 x 6 x 20-in. beams. Although further research is necessary, the indications are that the Griffith concept of a critical strain-energy-release rate being a condition for rapid crack propagation and consequent fracture, is applicable to concrete. The critical strain-energy-release rate may be ascertained by suitable analytical and experimental procedures and the fracture strength of concrete containing cracks can thereby be predicted.

■ THERE IS CONSIDERABLE EVIDENCE to show that microcracks occur in concrete when loaded. Evans[1] and Berg[2] used a microscope to detect microcracks on the tension face of beams in flexure and on the surface of concrete cylinders in uniaxial compression. These cracks were visible at a load considerably less than that required to cause fracture of the concrete. Blakey and Beresford[3] used electrical wire resistance strain gages to determine the onset of cracking of beams in flexure. Kaplan* applied a similar technique not only to beams in flexure but also to cylinders in direct and indirect tension. Jones[4], and Jones and Kaplan[5] have used an ultrasonic pulse velocity technique to indicate the development and orientation of microcracks in concrete cubes, uniaxially loaded, in compression. L'Hermite[6] has reported how the noise caused by cracking can be heard by means of a microphone.

Others have made similar observations which indicate the presence of cracks at a load very much less than that required to cause ultimate failure of the concrete. The initiation and propagation of these small

*Unpublished.

cracks are considered to have an important influence on the fracture strength of concrete. They may also be important in regard to other properties such at fatigue strength and creep.

The main purpose of the research reported in this paper was to ascertain whether the Griffith[7] concept of a critical strain-energy-release rate, is a necessary condition for rapid crack propagation and consequent fracture of concrete.

THE GRIFFITH THEORY

Notation

b = width of beam
c = notch depth
d = over-all depth of beam
h = $d - c$ = net depth of beam at notch
l = span length
r = radius of locus of Point A (see Fig. 1)
A = fracture area
E = Young's modulus
F = applied load per unit width of beam
G = strain-energy-release rate
G_c = critical strain-energy-release rate
K = $\sqrt{EG/\pi}$ = stress intensity factor
M = spring constant
M_b = bending moment at the notched section per unit width of beam
P = applied load
T = surface energy of the material per unit area
U = release of strain energy due to formation of the crack
W = work done in the course of crack formation
μ = Poisson's ratio
σ = stress
σ_n = nominal bending stress at root of the notch
δ = elastic displacement of the applied load

In 1920, Griffith[7] suggested that the fracture strength of a brittle material, such as glass, is greatly affected by the presence of small cracks which are either pre-existent or which are formed due to the application of a load. This effect, he said, was due to stress concentrations caused by these cracks. He also proposed a theory of fracture strength based on a consideration of changes in strain energy and surface energy as the largest of these cracks extends.

For an elliptical crack of length c through an infinite plate of unit thickness, so loaded that in regions remote from the crack it has a uniform tensile stress σ, normal to the plane of the crack, he deduced that

$$\frac{dU}{dA} = \frac{d}{dc}\left(\frac{\pi \sigma^2 c^2}{2E}\right) = \frac{\pi \sigma^2 c}{E} \quad \text{.....(1)}$$

and

$$\frac{dW}{dA} = \frac{d}{dc}(2Tc) = 2T \quad \text{.....(2)}$$

Griffith suggested that the crack would extend rapidly, and the specimen would rupture, when the rate of release of strain energy was at

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least equal to the rate of increase in free surface energy due to the formation of new surface area, i.e., $dU/dA = dW/dA$.

Therefore from Eq. (1) and (2)

$$\frac{\pi \sigma^2 c}{E} = 2T \quad \text{(3)}$$

and

$$\sigma = \sqrt{\frac{2ET}{\pi c}} \quad \text{(3a)}$$

which is the well-known Griffith formula.

MODIFICATION OF THE GRIFFITH THEORY

During World War II, interest in the Griffith theory of fracture strength was revived, mainly as a result of fracture failures in welded ships.[8] Since then failures of large oil-storage tanks, pressurized-cabin aircraft, and gas transmission lines have stimulated this interest, and the Griffith theory has been modified and extended to materials which are less brittle than glass.

Irwin[9] and Orowan[10] recognized that for materials in which plastic flow precedes failure, unstable crack extension can occur if the plastic strains tended to localize near the boundaries of the crack. They pointed out, however, that the Griffith theory must then be modified so as to take into account the dissipation of strain energy in plastic flow and suggested that the surface energy in Griffith's formula should be augmented by the work of plastic deformation. The view is now held that provided the surface energy term, in Griffith's formula, is replaced by the total work involved, or energy absorbed, in the process of fracture, then Griffith's theory in modified form may be applied to a wide range of materials. The strain-energy-release rate, dU/dA, which Irwin designates as G, may be less than that required to cause unstable crack propagation. This is because both the energy-release rate and the energy-absorption rate, during crack extension, may depend on several factors, such as the magnitude and rate of application of the load. The strain-energy-release rate at onset of unstable crack propagation is referred to as the critical strain-energy-release rate G_c.

Irwin[11-14] has further shown that for crack dislocations, particularly of the separational or opening-mode type, there is a close relationship between strain-energy-release rate and the intensity of the stress components in close vicinity to the edge of the crack. At a Point A, close to the leading edge of an internal free surface representing a crack (Fig. 1)

$$\sigma_y = \sqrt{\frac{EG}{2\pi r}} \cos\frac{\theta}{2}\left(1 + \sin\frac{\theta}{2}\sin\frac{3\theta}{2}\right) \quad \text{.....(4)}^*$$

The term $\sqrt{EG/\pi}$, designated as K, has been called the stress intensity factor. Irwin[11] has regarded the strain-energy release rate G, not only as a measure of the elevation of the stresses surrounding the crack extension but also as the driving force motivating crack extension. He has also suggested[13] that if two different load systems produce the same stress environment near the leading edge of a crack then the influence on crack extension should be similar. This indicates that the critical strain-energy-release rate G_c may be considered as being a fundamental property of a material in the same way as the modulus of elasticity is regarded.

CRITICAL STRAIN-ENERGY-RELEASE RATE DETERMINATION

Concrete is a visco-elastic, heterogeneous material in which failure can occur partly by fracture of the cement, fracture of the aggregate or by the failure of the bond between the cement and the aggregate. It is therefore unlikely that a consideration of surface energy alone will give a reliable estimate of the critical strain-energy-release rate G_c of concrete.

Two methods of determining G_c will be discussed.

Analytical method

One method, which will be referred to as the analytical method, is to use stress analysis procedures to derive a mathematical relationship for G in terms of the dimensions of the test specimen, depth of crack, applied load, and modulus of elasticity. A test is then done in which a gradually increasing load is applied to the test specimen until a stress level is reached which results in rapid propagation of the initial crack. G_c may then be calculated from the appropriate mathematical relationship.

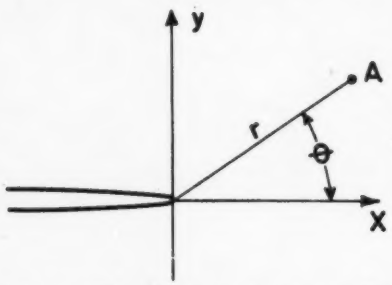


Fig. 1 — Coordinates at the leading edge of a crack

*Valid for plane stress only.

In this investigation the test specimens were concrete beams which had a sharp notch across the middle of the tension face to simulate the effects of a crack. The notches were of different depth and the beams were subjected to transverse or bending loads. Appropriate formulas for G have therefore to be obtained.

In the case of a semi-infinite plate of unit thickness, with a shallow edge crack of depth c, to which a tensile load is applied normal to the plane of crack extension, Griffith and others have shown that the strain-energy-release rate is given by

$$G = \frac{\pi \sigma^2 c}{E} \quad \text{.....(5)}$$

where σ = tensile stress normal to and remote from the crack.

For plane strain conditions E is replaced by $E / (1-\mu^2)$ where μ is Poisson's ratio.

Thus Eq. (5) becomes

$$G = \frac{(1-\mu^2)\pi \sigma^2 c}{E} \quad \text{.....(6)}$$

For rectangular beams, in which the notch depth c is shallow in relation to the over-all depth d, Eq. (6) may be used to determine G, the stress field σ being regarded as equivalent to the maximum tensile bending stress at the outermost fiber of the beam. It is also convenient to consider a beam of unit width.

Then

$$\sigma = \frac{6 M_b}{d^2} \quad \text{.....(7)}$$

and

$$\sigma_n = \frac{6 M_b}{h^2} \quad \text{.....(7a)}$$

From Eq. (7) and (7a)

$$\sigma^2 = \sigma_n^2 \frac{h^4}{d^4} \quad \text{.....(8)}$$

Substituting in Eq. (6)

$$G = (1-\mu^2) \frac{\sigma_n^2 h^4 \pi c}{E d^4} \quad \text{.....(9)}$$

As shown by Winne and Wundt,[15] Eq. (9) may be written in the form

$$G = \frac{(1-\mu^2)\sigma_n^2 h}{E} f\left(\frac{c}{d}\right) \quad \text{.....(10)}$$

where

$$f\left(\frac{c}{d}\right) = \frac{\pi c}{d}\left(1 - \frac{c}{d}\right)^3 \quad \text{.....(11)}$$

In regard to deeply notched beams, i.e., where c/d is large, Neuber[16] has obtained a solution for the notch stresses in pure bending, for a thin bar with a deep hyperbolic notch on one side only. The equation for the stress, normal to the plane of the crack at a distance r from the crack tip, when the radius at the root of the notch approaches zero (thus simulating a crack), becomes approximately[15]

$$\sigma = \frac{\sigma_n \left(\frac{\pi}{2} - 1\right)}{6\left(\frac{\pi^2}{8} - 1\right)\left(\frac{2r}{h}\right)^{1/2}} = \frac{0.406\,\sigma_n}{\left(\frac{2r}{h}\right)^{1/2}} \quad \ldots\ldots (12)$$

From Eq. (4), for $\theta = 0$

$$\sigma = \sqrt{\frac{EG}{2\pi r}} \quad \ldots\ldots (13)$$

Thus, from Eq. (12) and (13)

$$G = 0.52\,\frac{\sigma_n^2 h}{E} \quad \ldots\ldots (14)$$

For a condition of plane strain, Eq. (14) becomes

$$G = 0.52\,\frac{(1-\mu^2)\sigma_n^2 h}{E} \quad \ldots\ldots (15)$$

which is the same form as Eq. (10) with $f(c/d)$ being equal to 0.52.

Winne and Wundt[15] found that for moderate notch depths neither Eq. (10) nor (15) is applicable. They utilized a method described by Bueckner[17], to calculate G for notch depths ranging from 10 to 50 percent of the beam depth and ascertained that the results could be expressed in the form of Eq. (10) when appropriate values of $f(c/d)$ are used. These latter values are shown graphically in Fig. 2 together with those based on the shallow- and deep-notch formulas, Eq. (10) and (15), respectively. The indications are that the shallow-notch formula is suitable for c/d ratios which are less than 15 percent, and that the deep-notch formula is satisfactory for c/d ratios greater than 50 percent.

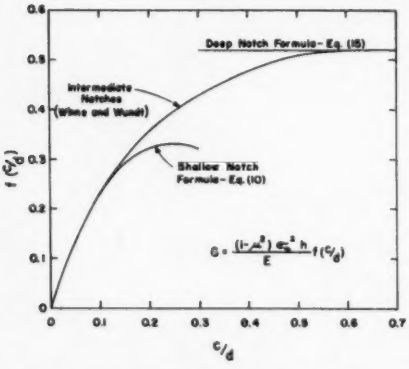


Fig. 2—Values of $f(c/d)$ for different c/d ratios

The strain-energy-release rate for beams with different notch depths may be calculated from an equation in the form of Eq. (10), the appropriate $f(c/d)$ values being obtained from Fig. 2.

TABLE 1 — MIX PROPORTIONS

Mix	Type of coarse aggregate	Proportions, by weight				Average slump, in.
		Cement	Sand	Coarse aggregate	Water	
1	—	1	2.40	—	0.50	7.5
2	Crushed limestone	1	2.40	2.67	0.50	0.2
3	Quartzite gravel	1	2.40	3.60	0.60	5.8

Direct experimental method

Irwin and Kies[18, 19] suggested another method, here referred to as the direct experimental method, in which the change in the spring constant with crack extension, can be used to measure the release of the elastic energy G.

If, by definition

$$F = M\,\delta \qquad (16)$$

it may be shown[18] that

$$G = \frac{1}{2} F^2 \frac{d}{dc}\left(\frac{1}{M}\right) \qquad (17)$$

Using load-deflection measurements the reciprocal of the spring-constant $1/M$, may be determined for beams with different notch depths. As discussed below, values of $(d/dc)\,(1/M)$ for different depths of notch, can be obtained by graphical or other means. G is then calculated from Eq. (17).

EXPERIMENTAL DETAILS

Materials and mix proportions

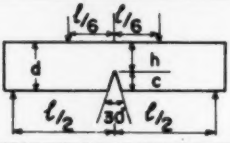
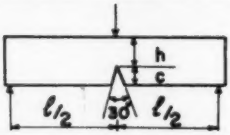
Type I portland cement was used. It was purchased in a single lot on the open market, blended for uniformity and then stored in airtight steel drums. The fine aggregate was a siliceous sand which had a fineness modulus of approximately 2.6. The crushed limestone and quartzite gravel, which were used as coarse aggregate, were separated into ¾ to ½ in., ½ to ⅜ in., and ⅜ in. to No. 4 sizes, and recombined in equal proportions. All aggregates were air dried.

As shown in Table 1, three different mixes were investigated. In each mix the ratio of cement to sand, by weight, was the same. All specimens were prepared in accordance with relevant ASTM methods.

Test specimens

Specimens of two different sizes, 3 x 4 x 16-in. and 6 x 6 x 20-in., were made for testing as beams. As indicated in Table 2, three out of four beams, for each test condition, contained a sharp, uniform notch across the middle of one side. The notches, which had an included angle of 30 deg, differed in depth and were formed by fixing the appropriate size of steel wedge to a vertical side of the

TABLE 2 — DIMENSIONS OF BEAMS TESTED

Third-point loading				Center-point loading			
							
Beam size, in.	c, in.	d, in.	l, in.	Beam size, in.	c, in.	d, in.	l, in.
3 x 4 x 16	0 0.5 1.0 1.5	3	12	3 x 4 x 16	0 0.5 1.0 1.5	3	12
6 x 6 x 20	0 1.0 2.0 3.0	6	18	6 x 6 x 20	0 1.0 2.0 3.0	6	18

mold before casting the test specimens. The specimens were made and moist cured in accordance with the appropriate ASTM procedure except that Mix 2, a low slump concrete, was consolidated with an internal vibrator.

Sufficient material was mixed each day to make four 3 x 4 x 16-in. beams, one without a notch and the others with notch depths of 0.5, 1, and 1.5 in. and four 6 x 6 x 20-in. beams, one without a notch and the other three with notch depths of 1, 2, and 3 in. This was repeated on six different days for each mix at random, making a total of 144 test specimens.

Test procedures

The beams were tested 28 days after they were made. Shortly before the time of test the specimens were removed from the fog room and kept moist by wet burlap covers until tested. For every beam loaded at the third points, another was loaded at the center of the loading span until fracture occurred. The rate of loading for all beams was such that, in the case of the corresponding unnotched specimens, the maximum outer fiber stress increased at the rate of 150 psi per min. Thus for the 3 x 4 x 16-in. beams, both with and without a

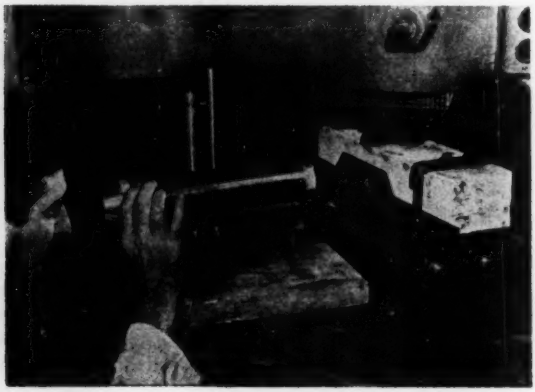


Fig. 3 — General arrangement for load-deflection tests

notch, the loading rate was 450 lb per min for third-point loading and 300 lb per min for center-point loading. The 6 x 6 x 20-in. beams were loaded at rates of 1800 and 1200 lb per min. For each mix, beam size, depth of notch and method of loading, a test was made on each of three different days.

Concrete cylinders 12 in. long and 6 in. in diameter were also made to determine Poisson's ratio in accordance with ASTM Test Method C 215.[20] The static modulus of elasticity was determined* from load-strain measurements on unnotched beams, the strain being obtained by means of electrical wire resistance strain gages fixed to the tension face.

TABLE 3—MODULUS OF ELASTICITY, E, AND POISSON'S RATIO, μ, OF THE CONCRETE

Mix	E, psi $\times 10^{-6}$	μ
1	4.05	0.20
2	5.47	0.21
3	4.19	0.20

TABLE 4 — VALUES OF $f(c/d)$

c/d	$f(c/d)$
0.17	0.32
0.33	0.45
0.50	0.51

Load-deflection measurements were made to ascertain the reciprocal of the spring-constant $1/M$. Because the maximum deflection is approximately 2×10^{-4} in., high precision measurements were necessary. For third-point loading, the method used was in accordance with that described by Mong and Pendergast.[21] In essence, a Tuckerman optical strain gage was used to determine the deflection, at the center of the beam, in relation to a reference frame which rested on the specimen over the supports. The displacement is indicated on the scale of an autocollimator, which is a specially designed precision telescope (Fig. 3). For center-point loading, their method was modified as shown in Fig. 4. Using a 2-in. Tuckerman gage with a 0.2 in. lozenge, the deflection of the specimen was the same as the displacement indicated on the scale of the autocollimator. The sensitivity of the deflectometer was 2 microin. and the accuracy has been given[21] as ± 4 microin. Deflection readings were made at 50-lb load intervals in tests on the 3 x 4 x 16-in. beams and at 200-lb load intervals on the 6 x 6 x 20-in. beams until the beams fractured.

*Previously referred to work of the author, as yet unpublished.

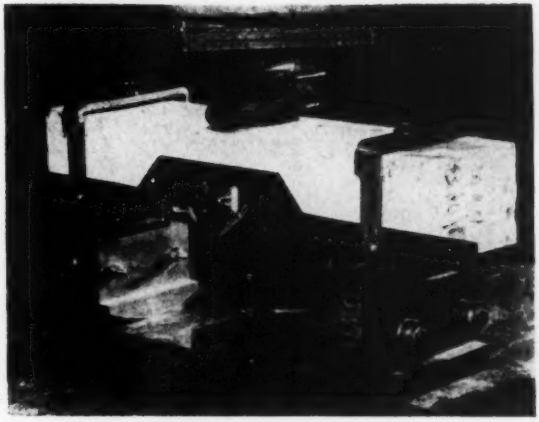


Fig. 4—Deflectometer for deflection measurements on 3 x 4 x 16-in. beams loaded at the center

TEST RESULTS

Analytical method

The average moduli of elasticity, and Poisson's ratios, for the three mixes, are given in Table 3. The values of $f(c/d)$, given in Table 4, were obtained from Fig. 2 which was constructed by using Eq. (10) and (15) in the appropriate ranges, and the values given by Winne and Wundt.[15] These values of $f(c/d)$ are based on the assumption that there is no crack extension before fracture. Average values of F at failure, and the critical strain-energy-release rates G_c, calculated from an equation in the form of Eq. (10), appear in Table 5.

Direct experimental method

From Eq. (16)

$$\frac{1}{M} = \frac{\delta}{F} \qquad (18)$$

Since $F = P/b$, where P is the applied load and b the width of the beam

$$\frac{1}{M} = \frac{\delta b}{P} \qquad (19)$$

As shown in Fig. 5, the value of $1/M$ was obtained by plotting load P against deflection δ, and multiplying the reciprocal of the slope of the initial straight line portion of the curve by the width of the beam. The values of $1/M$ given in Table 6, are the averages of two, and in some cases three, tests on different specimens. The reproducibility was such that the results of repeat tests seldom differed by more than 5 percent.

To obtain the rate of change in $1/M$ with crack depth c, i.e., $(d/dc)(1/M)$, $1/M$ may be plotted against c (or c/d), as has been done in Fig. 6,

TABLE 5 — VALUES OF F AT FAILURE AND CRITICAL STRAIN-ENERGY-RELEASE RATES*

Size of beam, in.	Method of loading	c, in.	c/d	Mix 1		Mix 2		Mix 3	
				F, lb per in.	G_c, lb per in.	F, lb per in.	G_c, lb per in.	F, lb per in.	G_c, lb per in.
3 x 4 x 16	Third-point	0.5	0.17	334	0.079	394	0.080	313	0.066
		1.0	0.33	214	0.088	248	0.087	178	0.059
		1.5	0.50	123	0.078	147	0.082	109	0.058
3 x 4 x 16	Center-point	0.5	0.17	242	0.092	285	0.094	207	0.065
		1.0	0.33	151	0.098	188	0.112	122	0.062
		1.5	0.50	87	0.087	108	0.099	78	0.068
6 x 6 x 20	Third-point	1.0	0.17	758	0.112	917	0.122	680	0.088
		2.0	0.33	473	0.121	639	0.161	451	0.106
		3.0	0.50	274	0.108	375	0.150	254	0.091
6 x 6 x 20	Center-point	1.0	0.17	575	0.145	744	0.181	462	0.092
		2.0	0.33	357	0.155	465	0.193	330	0.127
		3.0	0.50	209	0.142	253	0.155	179	0.101

*G_c calculated from Eq. (10) using appropriate $f(c/d)$ values.

TABLE 6—VALUES OF $1/M$, $(d/dc)(1/M)$, AND CRITICAL STRAIN-ENERGY-RELEASE RATES*

Size of beam, in.	Method of loading	c, in.	c/d, in.	Mix 1			Mix 2			Mix 3		
				$\frac{1}{M}$, sq in. per lb $\times 10^6$	$\frac{d}{dc}\left(\frac{1}{M}\right)$, in. per lb $\times 10^6$	G_c, lb per in.	$\frac{1}{M}$, sq in. per in. $\times 10^6$	$\frac{d}{dc}\left(\frac{1}{M}\right)$, in. per lb $\times 10^6$	G_c, lb per in.	$\frac{1}{M}$, sq in. per lb $\times 10^6$	$\frac{d}{dc}\left(\frac{1}{M}\right)$, in. per lb $\times 10^6$	G_c, lb per lb
3 x 4 x 16	Third point	0	—	3.50	0	—	2.37	0	—	2.88	0	—
		0.5	0.17	3.87	1.77	0.099	2.72	1.27	0.098	3.20	1.64	0.080
		1.0	0.33	5.60	5.52	0.126	3.76	3.26	0.100	4.83	5.14	0.081
		1.5	0.50	9.80	11.75	0.082†	6.60	8.93	0.096†	8.54	9.93	0.059†
3 x 4 x 16	Center-point	0	—	4.10	0	—	2.87	0	—	3.70	0	—
		0.5	0.17	4.52	2.12	0.062	3.30	1.85	0.075	4.04	1.79	0.038
		1.0	0.33	6.72	7.26	0.082	4.93	4.96	0.087	6.00	6.63	0.049
		1.5	0.50	12.40	16.12	0.061†	8.62	10.23	0.060†	11.10	15.42	0.047†
6 x 6 x 20	Third-point	0	—	1.55	0	—	1.14	0	—	1.27	0	—
		1.0	0.17	1.84	0.57	0.163	1.36	0.37	0.156	1.52	0.53	0.122
		2.0	0.33	2.76	1.35	0.151	1.92	0.84	0.171	2.40	1.28	0.130
		3.0	0.50	4.80	2.88	0.107†	3.40	2.34	0.164†	4.20	2.41	0.078†
6 x 6 x 20	Center-point	0	—	1.84	0	—	1.30	0	—	1.62	0	—
		1.0	0.17	2.30	0.93	0.153	1.62	0.57	0.157	2.02	0.55	0.058
		2.0	0.33	3.72	1.93	0.123	2.49	1.31	0.142	3.00	1.14	0.062
		3.0	0.50	6.30	3.16	0.069†	4.72	3.48	0.112†	5.92	4.05	0.065†

*G_c calculated from Eq. (17) using values of F taken from Table 5.
†Values not considered reliable because of manner in which $(d/d_c)(1/M)$ was determined.

and the slope of the curve determined graphically. This procedure is not considered to be very accurate.

The method adopted was to fit a polynomial, in terms of crack depth c, to the experimentally determined values of $1/M$. The relationship between $1/M$ and c was assumed to be

$$\frac{1}{M} = A + Bc^2 + Cc^3 + Dc^4 \quad \text{(20)}$$

where A, B, C, and D are constants. Thus

$$\frac{d}{dc}\left(\frac{1}{M}\right) = 2Bc + 3Cc^2 + 4Dc^3 \quad \text{(21)}$$

In Eq. (20) there is no term in c to the first power because when $c = 0$, $(d/dc)(1/M) = 0$. The constants are obtained by substituting the experimentally determined values of $1/M$, for different crack depths, in Eq. (20).

The values of $(d/dc)(1/M)$ calculated from Eq. (21) are given in Table 6. It must, however, be mentioned that the curve represented by Eq. (20), although passing through the experimental points, is not necessarily smooth, as an mth degree polynomial can be made to pass through any $m+1$ points in a plane. It is therefore possible that the derivatives obtained from Eq. (21) may not be entirely accurate. This applies particularly to c/d ratios of 0.50 and the value of the derivative at this ratio is not considered to be reliable. In calculating G_c from Eq. (17), the $(d/dc)(1/M)$ values used were also based on the assumption that there is no crack extension before fracture. This may be an incorrect assumption in view of the nonlinearity of the $P - \delta$ curves as shown in Fig. 5.

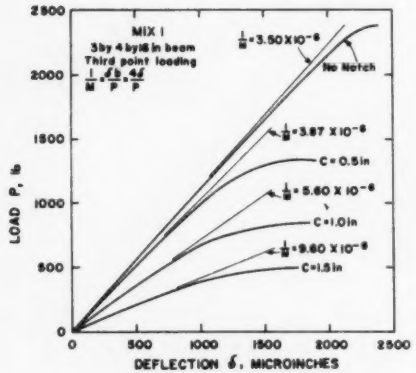


Fig. 5—Determination of $1/M$

DISCUSSION OF RESULTS

Depth of notch

In Fig. 7-9, the G_c values obtained from tests on beams of the same size, and subjected to the same method of loading, have been compared for different c/d ratios. As shown in Table 7, the coefficients of variation between the average G_c values, for beams with different notch depths, were 6.0, 4.6, and 4.5 percent for Mixes 1, 2, and 3 respectively. This represents

excellent agreement for a nonhomogeneous material like concrete. For c/d ratios of 0.17 and 0.33, the agreement between G_c values obtained by the direct experimental method was comparable with that obtained by the analytical method. Because G_c did not vary greatly for beams with different notch depths, the averages have been used in the following discussion. In the experimental method the averages do not include the values for beams with c/d ratios of 0.50.

TABLE 7—VARIATION IN G_c FOR DIFFERENT NOTCH DEPTHS

	G_c, lb per in.		
c/d	Mix 1	Mix 2	Mix 3
0.17	0.113	0.121	0.077
0.33	0.117	0.131	0.084
0.50	0.104	0.121	0.079
Average G_c	0.111	0.124	0.080
Coefficient of variation, percent	6.0	4.6	4.5

Method of loading

Values of G_c, obtained from third-point and center-point loading tests on the same size of beam, have been compared in Fig. 10. It will be seen that for G_c ascertained by the analytical procedure, the two methods of loading did not give greatly different results. On the average, third-point loading gave 15 percent lower values of G_c than did center-point loading. The direct experimental method produced somewhat more divergent and scattered results, center-point loading giving, on the average, 26 percent lower values than third-point loading.

Size of test specimen

In Fig. 11, G_c values from tests on 3 x 4 x 16-beams are compared with those from tests on 6 x 6 x 20-in. beams similarly loaded. In both the analytical and direct methods the smaller beams gave consistently lower values, being on the average 38 percent less than the values obtained from the larger beams.

As discussed below, slow crack growth prior to instability, may account for these somewhat large discrepancies. In the tests on the 6 x 6 x 20-in. beams, the span was only three times the depth of the

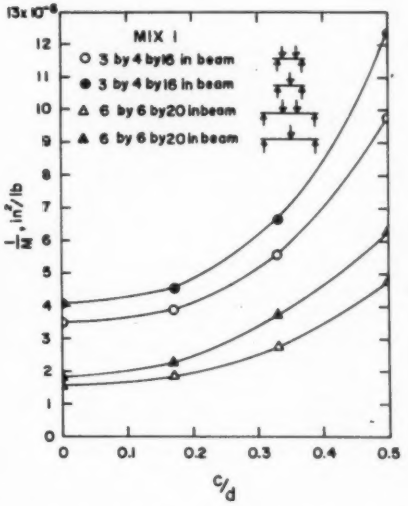


Fig. 6—$1/M$ plotted against c/d

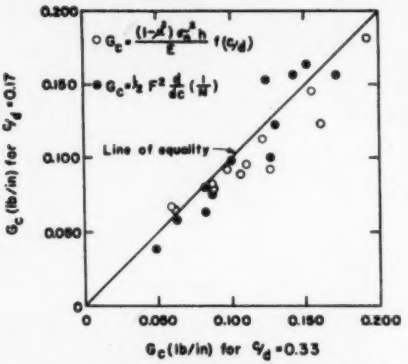


Fig. 7—Comparison of G_c for c/d ratios of 0.17 and 0.33

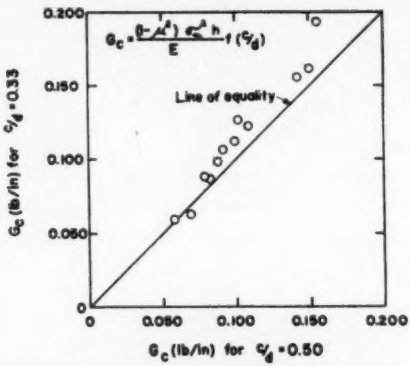


Fig. 8—Comparison of G_c for c/d ratios of 0.33 and 0.50

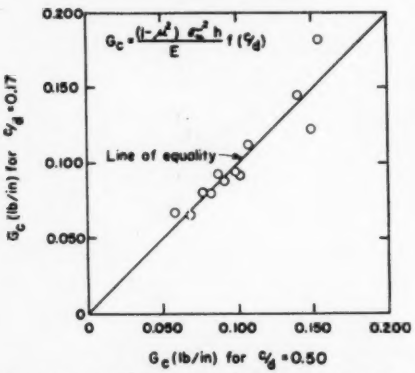


Fig. 9—Comparison of G_c for c/d ratios of 0.17 and 0.50

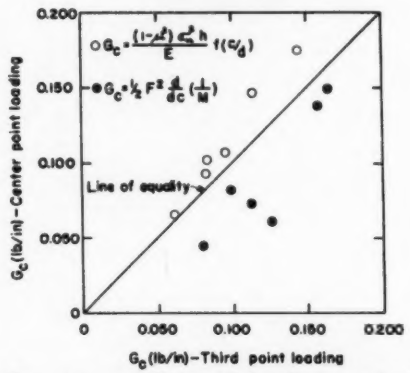


Fig. 10 — Effect of loading method on G_c

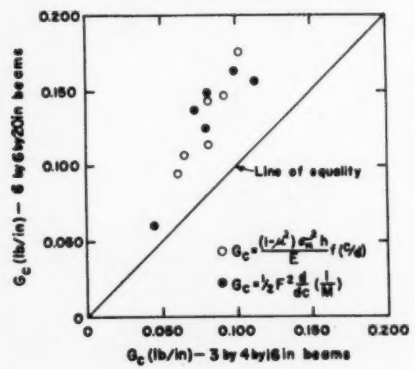


Fig. 11—Effect of specimen size on G_c

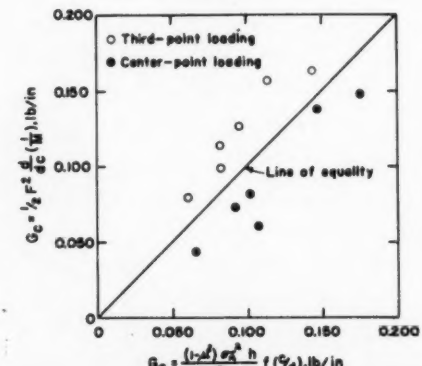


Fig. 12—Comparison of G_c obtained by analytical and direct experimental methods

beam. For the 3 x 4 x 16-in. beams the span-depth ratio was 4. The effect of shear may therefore also be important and requires further investigation.

In Fig. 12, the G_c values obtained by the analytical and direct experimental methods have been compared for beams of the same size and similarly loaded. For third-point loading, the analytical method values are, on the average, 21 percent less than the direct method values, while for center-point loading the reverse was true, the direct method values being 21 percent less than the analytical method values.

Slow crack extension

In determining G_c, the depth of the crack at instability was assumed to be the depth of the notch. This assumption may be incorrect in that the notch depth may increase prior to instability due to slow crack growth at the tip of the notch as the load is increased. The effect of this would be to increase the G_c values which have been calculated.

A staining technique has been suggested[22,23] to ascertain the extent of slow crack growth prior to fracture. A small quantity of staining fluid is placed in the notch root just prior to loading. As the crack extends the fluid penetrates into the crack and stains the fracture surface. When the crack depth increases rapidly the stain does not follow and the stained fracture surface may only indicate the extent of slow crack growth.

This technique was tried on some 3 x 4 x 16-in. beams tested to failure in the usual way, with the notch at the bottom. The root of the notch was kept continuously wet with black India ink during loading. On examining the fracture surfaces it was evident that slow crack growth had taken place prior to instability. The depth of the crack may also be inferred by determining the deflection-load ratio, i.e., $1/M_c$, at fracture. This value may then be used to obtain the final crack depth from a graph showing the relationship between $1/M$ and c.

It is possible that the extent of slow crack growth, for a given concrete mix, may be affected by factors such as the rate of loading and the size of the test specimens. Crack depth correction procedures may well minimize the differences in the G_c values obtained in this investigation. If, for example, the relative extent of slow crack growth is greater for small specimens than for large specimens it would reduce the differences in G_c obtained from the tests on 3 x 4 x 16-in. and 6 x 6 x 20-in. beams.

Further work is required to investigate the extent to which slow crack growth is affected not only by specimen size and the rate and method of loading but also by variations in mix proportions, type of aggregate used and curing conditions. Similar studies regarding the velocity of crack propagation may also prove to be fruitful.

G_c and the surface energy of concrete

Brunauer[24] has found the surface energy of tobermorite, which constitutes about 75 percent of hydrated portland cement by weight, to be approximately 390 ergs per sq cm. The surface energy of quartz[25] is approximately 980 ergs per sq cm.

In Mix 1, the ratio of cement to quartzite, by volume, is about 1 to 3. On the assumption that no bond failure occurs and that the fracture surface is smooth, an approximate estimate of the surface energy of this mix is

$$T = \frac{[390 + (3 \times 980)]}{4} = 830 \text{ ergs per sq cm}$$

$$= 0.0046 \text{ in.-lb per sq in.}$$

$$2T = 0.0092 \text{ in.-lb per sq in.}$$

The Griffith theory for brittle materials suggests that fracture will occur when the strain-energy-release rate just exceeds twice the surface energy of the material. The average G_c value for Mix 1 concrete was 0.11 in. lb per sq in. which is approximately 12 times greater than the value of $2T$ obtained above. It is interesting to note that the G_c of glass is approximately ten times greater than twice its solid state surface energy.[26]

Practical application

Although further investigations are necessary, it is not considered premature to suggest possible practical applications of the results of the procedures described in this report. The quantitative determination of the load required to cause fracture of a specimen which is cracked, immediately suggests itself. A simple example will be used to illustrate this.

Consider a flat slab of plain concrete, 4 in. thick, simply supported on a span $l = 24$ in. and carrying a uniformly distributed load w psi. Examination reveals that there is a crack across the bottom of the slab at the middle of the span. The average depth of the crack is 0.5 in. What approximately is the maximum load that this slab can support?

Let it be assumed that tests on notched beams, made of concrete similar to that used in the slab, give a G_c value of 0.12 in.-lb per sq in. Young's modulus $E = 5.5 \times 10^6$ psi and Poisson's ratio $\mu = 0.20$. From Eq. (7a)

$$\sigma_n = \frac{6 M_b}{h^2} = \frac{6 w l^2}{8 h^2}$$

For $c/d = 0.125$, the value of $f(c/d)$ obtained from Fig. 2 is 0.25. Substituting these values in Eq. (10) the maximum load to cause fracture is found to be 25 psi.

The previous example illustrates how the load necessary to cause fracture failure may be predicted from measurements of the critical strain-energy-release rate together with a knowledge of the size and location of the starting crack.

CONCLUSIONS

The Griffith concept of strain-energy-release rate is related to the locally elevated stress field in the vicinity of a crack in a material under load. Stress analysis procedures were used in conjunction with tests on notched beams to obtain estimates of the critical strain-energy-release rate of concrete. The results indicate:

1. Beams with different depths of notch, gave G_c values which are in close agreement, the average coefficient of variation being 5 percent.

2. Third-point loading gave, on the average, 15 percent lower values of G_c than center-point loading.

3. G_c values for 3 x 4 x 16-in. beams were, on the average, 38 percent less than those obtained from tests on 6 x 6 x 20-in. beams.

A direct experimental method gave G_c values which, on the average, differed by 21 percent from those obtained by the analytical method.

Further research is necessary, particularly in regard to the question of slow crack growth prior to fast fracture. The results of this investigation nevertheless indicate that the Griffith concept of a critical strain-energy-release rate being a condition for rapid crack propagation and consequent fracture, is applicable to concrete. The critical strain-energy-release rate may be ascertained by suitable analytical and experimental procedures and it is possible that the fracture strength of concrete containing cracks may thereby be predicted.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

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Propagación de la Grieta y Fractura del Hormigón

Se discute en este estudio la aplicación de la teoría de la Grieta de Griffith a la resistencia a la fractura.

Se realizaron ensayos en vigas de hormigón con grietas simulando nudos, usándose dos métodos, uno denominado método analítico y el otro método directo experimental, los cuales se usaron para determinar la razón de la liberación de la energía en la deformación crítica G_c asociada con la expansión rápida de la grieta.

Hubo buen acuerdo entre los valores G_c para vigas con diferentes profundidades de nudos, las que fueron cargadas por los métodos de los puntos tercios y del punto central. Sin embargo, las vigas de 3 x 4 x 16-pulgadas dieron valores G_c algo superiores que las de 6 x 6 x 20-pulgadas.

Aunque es necesario mayor número de ensayos, quedó demostrado que el concepto Griffith de la razón de la liberación de la energía en la deformación crítica es una condición para la propagación rápida de la grieta y la fractura consecuente. La razón de la liberación de la energía en la deformación crítica puede encontrarse por procedimientos analíticos y experimentales, y en consecuencia, puede predecirse la resistencia a la fractura del hormigón conteniendo grietas.

Propagation de Fissures et la Rupture de Béton

On discute la théorie Griffith de fissures, sur la résistance à la rupture. On a fait des essais sur poutres de béton pourvues des entailles pour simuler les fissures, et on a employé deux méthodes, dites la méthode analytique et la méthode directe expérimentale, pour déterminer le taux limite G_c de déformation-énergie-détente associé à l'extension rapide de la fissure. Il y avait une bonne concordance entre les valeurs de G_c pour les poutres ayant des entailles à profondeurs différentes et chargées par les méthodes dites à troisième point et à pivot. Cependant, les poutres de 3 x 4 x 16-pouces donnaient des valeurs de G_c un peu plus grandes que celles des poutres de 6 x 6 x 20-pouces. Les études ne sont pas complètes mais il y a toute apparence que la théorie de Griffith, qu'un taux limite de déformation-énergie-détente soit une condition de la propagation rapide de fissures et de la rupture qui en résulte, s'applique au béton. Le taux limite de déformation-énergie-détente peut être déterminé par des procédés analytiques et expérimentaux convenables, et l'on peut ainsi prédire la résistance à la rupture de béton fêlé.

Rissfortpflanzung und der Bruch bei Beton

Die Risstheorie von Griffith über Bruchfestigkeit wird diskutiert. Versuche an, mit Rissen ähnelnden künstlichen Einkerbungen versehenen Betonträgern wurden durchgeführt. Zwei Methoden, die die analytische und die direkte experimentelle Methode genannt worden sind, wurden dazu benutzt, die kritische Auslösungsgeschwindigkeit G_c der Spannungsenergie zu bestimmen, die mit der schnellen Verlängerung des Risses in Beziehung steht. Es bestand gute Übereinstimmung zweischen den G_c-Werten für Träger mit verschiedenen Einkerbungstiefen, die sowohl nach der Drittelspunkt- als auch nach der Mittelpunktmethode belastet waren. Jedoch ergaben 3 x 4 x 16 -Zoll Träger etwas grössere G_c-Werte als 6 x 6 x 20 -Zoll Träger. Obwohl weitere Untersuchungen notwendig sind, bestehen Anzeichen dafür, dass die Auffassung von Griffith, dass eine kritische Auslösegeschwindigkeit der Spannungsenergie eine Voraussetzung für schnelle Rissausbreitung und folgendem Bruch ist, für Beton anwendbar ist. Die kritische Auslösegeschwindigkeit der Spannungsenergie kann durch geeignete analytische und experimentelle Verfahren ermittelt und die Bruchfestigkeit von Risse enthaltendem Beton somit vorausgesagt werden.

Title No. 58-29

Influence of Sand Concentration on Deformations of Mortar Beams Under Low Stresses

By O. ISHAI

Slender mortar beams, prepared with different sand concentrations, were subjected to a series of bending tests consisting of instantaneous loading and unloading, to prolonged creep tests, and to strength and density tests, to establish the influence of the volume concentration of the sand on the rheological parameters of the mortar.

Results show that up to a volume concentration of 50 percent, rigidity, density, and strength increase slightly and almost linearly with sand concentration. In addition, the stress-strain relationship is also linear in this range up to failure.

On the other hand, at sand concentrations above 60 percent, a steep decrease was observed for the above properties with increasing concentration as well as deviation from linearity of the stress-strain curve at about 60 percent of ultimate stress. Most of the values reach a maximum in the intermediate range of 50-60 percent.

■ INCREASING THE SAND CONTENT of a mortar leads to a state in which the sand particles come into mutual contact. The theoretical volume concentration c_v (volume ratio of the solid particles to the total) corresponding to this "loose packing" stage is 52.4 percent, assuming spherical and uniform sand particles. It is probable that this concentration leads to a qualitative change in the properties of the mortar, conversion from a brittle to a plastic material differing in intergranular friction, a factor exerting considerable influence on the mechanical behavior of the material.

Compressive tests up to failure, carried out on mortar cylinders of different concentrations,[1] showed that in the 50-60 percent range, the properties of the material actually undergo a change. Accordingly, mortar can be classified into two distinct types: (1) brittle mortar at low sand concentrations (below 50 percent) failing by separation, and (2) nonbrittle or plastic mortar at high concentrations (above 60 percent) failing by sliding.

The object of the present study is an analysis of the deformational behavior of the two types, and of the influence of the sand content on their parameters.

PREPARATION AND CURING OF MORTARS

For purposes of comparison, the specimens used were slender beams (32.9 x 0.86 x 0.86 in.) similar to those used in previous studies of the rheological behavior of hardened cement paste.[2-3]

The ingredients were normal portland cement and standard Leighton Buzzard sand. A uniform water-cement ratio of 0.32 was used in all mixes because quartz sand absorbs no water, therefore all of the mixing water is allowed to participate in the hydration process. A uniform water-cement ratio thus results in a cement paste of constant composition and strength, irrespective of the sand content of the mortar.

A total of 104 beams were cast, comprising 26 batches, four specimens per batch. The sand concentration varied from zero to 80 percent, in 4 percent intervals. After identical mixing and compacting, the molds were stored in an insulated chamber. The specimens were demolded after 24 hr and wrapped in a thin nylon sheets as a means of uniform retention curing, and also for eliminating the effects of humidity fluctuations during the experiments. The beams were stored in the same chamber for 28 days under controlled and constant conditions of 16 C (61 F) and a relative humidity of 75-80 percent.

DESCRIPTION OF TESTS

The first group of specimens comprising 22 batches (covering the entire range of concentration) were subjected to the following tests:

Ultrasonic test

The time of propagation of an ultrasonic wave along the beam was measured (readings to 10^{-6} sec) by means of a Cawkell ultrasonic tester designed by the Road Research Laboratories, England.

Instantaneous loading tests

Each specimen was placed on two supports and subjected to a concentrated load at midspan. Its deflection at the point of application was measured by a dial gage to the nearest 10^{-4} in. Immediately following the zero reading, the lowest load was applied and immediately removed, with readings taken on application and removal. After a short interval, sufficient for full recovery of whatever time-deflection there was, a third reading was taken, and a higher load was applied and immediately removed, with two readings taken again, and so on up to fracture. The loads at fracture were noted. The actual time of application was limited to the minimum necessary for the readings to eliminate, as far as possible, any time-dependent deflection.

Bulk density

Immediately on failure, two pieces were taken from each beam, and subjected to a series of weighings in air and in water to determine the exact bulk density of the material.

O. Ishai, instructor in the Division of Mechanics at Technion, Israel Institiute of Technilogy, Haifa, has been studying the creep of concrete for the last 5 years. Mr. Ishai, with Joseph Glucklich, wrote "Rheological Behavior of Hardened Cement Paste under Low Stresses" which appeared in the Feb. 1961 JOURNAL.

Creep tests

A second group of specimens comprising 14 beams (seven with a sand concentration of 36 percent and seven of 68 percent) was subjected to creep tests at 28 days in the insulated chamber under the same constant temperature as before. These tests were on the line of those conducted by Glucklich.[2]

Under constant load — Four specimens for each concentration were placed horizontally on two supports and subjected to midspan loads, giving maximum flexural moments equivalent to M_o, $2M_o$, $4M_o$, and $6M_o$, M_o being the midspan moment due to the dead load of the beam. The loads were allowed to act for about 130 days. Midspan deflection readings were taken on alternate days on a stationary measuring device, care being taken to maintain the same position of the beam at each reading.

Under periodical loading — Six beams, three for each concentration, were tested with the aid of the device designed by Glucklich, for testing hardened cement paste (see Fig. 1).[3] The beams were subjected to horizontal loads at midspan, and deflections were also measured horizontally, so as to eliminate the effect of the dead load and permit complete unloading. Each set of three beams was subjected to moments equivalent to $2M_o$, $4M_o$, and $6M_o$ as above. The loads were allowed to act for 1 week and then removed for about 3 weeks, so as to allow the recovery process to end, and then reapplied for the next cycle. Each beam underwent four cycles as above, and daily dial gage readings were taken.

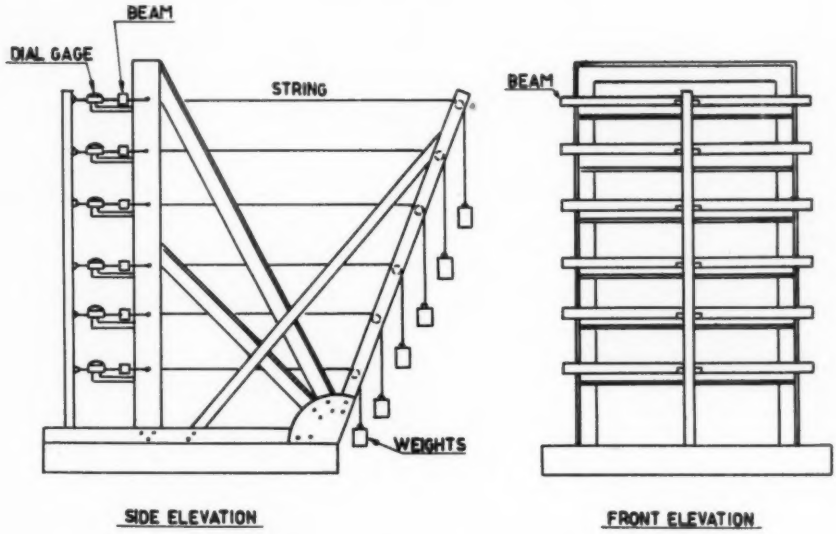

Fig. 1—Loading device for beams in flexure

RESULTS AND ANALYSIS*

Velocity of the ultrasonic wave

The dependence of the wave velocity on the sand concentration is shown in Fig. 2; a slow and almost linear increase up to a maximum at $c_v = 52$ percent and a steep decrease above it.

Instantaneous loading tests

Load-deflection—The differences between the readings on loading and unloading gave the elastic instantaneous deflection, while the differences between the readings preceding loading and the zero reading gave the residual instantaneous deflection (permanent set). Fig. 3 and 4 represent load-deflection curves up to failure for different concentrations. They show a fundamental difference of behavior between the lower concentrations (below 50 percent) which are linear up to failure, and the higher concentrations characterized by a deviation from linearity beginning at about 60 percent of the ultimate load.

Breaking stress—In spite of the high degree of scatter in the ultimate stress versus sand concentration curve (Fig. 2) a definite trend can be seen, namely, an increase in strength up to $c_v = 36$ percent, and a clear steep decrease at higher concentrations, up to 1/5 of the maximum at $c_v = 80$ percent.

Bulk density

The bulk density curve follows the same pattern (Fig. 5), a slow and almost linear increase up to a maximum of 50-52 percent, and a steep decrease above it. The constant-slope curve represents the theoretical variation of the bulk density assuming no air voids. This curve which almost coincides with the experimental curve below 50 percent shows that in that range the mortar is closely packed, and its void ratio is extremely low.

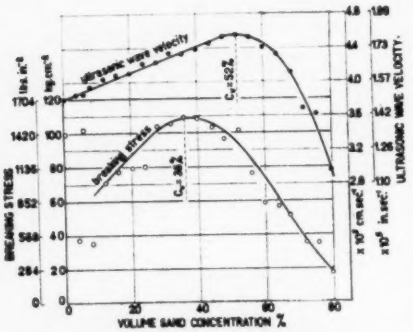

Fig. 2—Ultrasonic wave velocity and breaking stress versus sand concentration for mortar beams in flexure

On the other hand above 52 percent the voids ratio increases steadily and reaches a maximum of 23 percent at $c_v = 80$ percent.

Creep test under constant load

The six creep curves obtained for the two types of mortar under three load levels (Fig. 6) show that both the deflection proper and its time rate are higher for the mortar with the higher sand con-

*All data are the means of four specimens for each concentration. The standard deviations are relatively low, except for the breaking loads.

centration. This, however, has no comparative significance, since the two types differ in strength.

Creep tests under periodic loading

Results of this test are presented in Fig. 6 ($M_o = 8.25$ in.-lb) by two curves for the two types of mortar. They show how the creep varies from one cycle to another. The curves together with Table 1, in which the over-all deflections are resolved into their respective components, lead to the following preliminary conclusions:

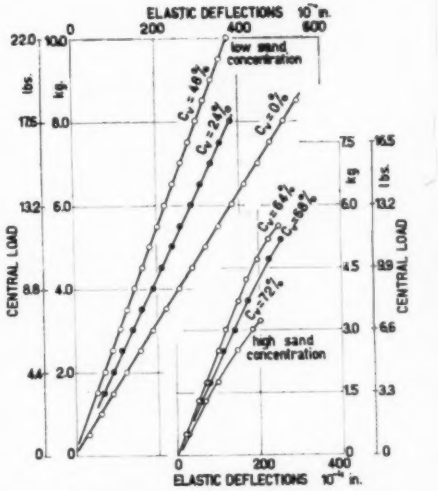

Fig. 3—Load versus instantaneous elastic deflections for mortar beams in flexure loaded to fracture

1. Recovery was practically completed within 3 weeks after unloading.

2. Instantaneous elastic deformation varied little, but actually decreased from one cycle to another.

3. Unloading curves show a seemingly exponential behavior of reversible creep which tends to a horizontal asymptote.

4. Reversible creep decreases from cycle to cycle.

5. Nonreversible creep increases in most cases, but at a reduced rate from cycle to cycle.

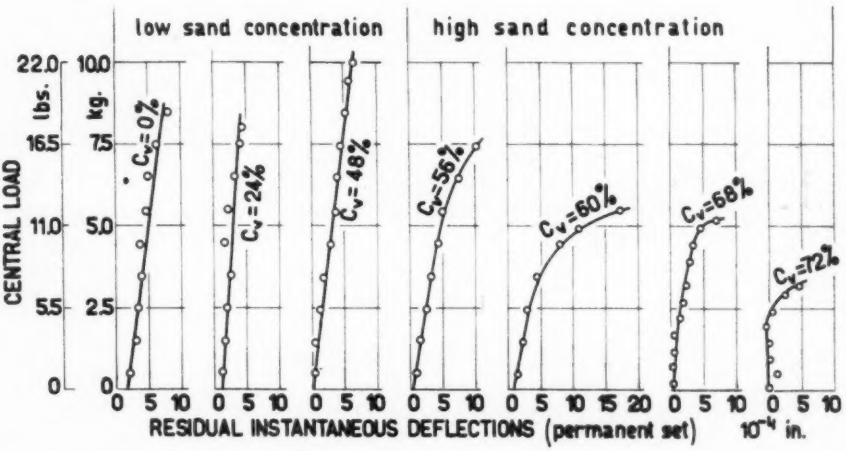

Fig. 4—Load versus permanent set for mortar beams in flexure loaded to fracture

QUANTITATIVE ANALYSIS AND RHEOLOGICAL PARAMETERS

Notation

c_v = volume concentration of sand
l = span length
E = Young's modulus
E_m = Young's modulus for mortar
E_p = Young's modulus for hardened cement paste
I = moment of inertia
G = modulus of rigidity
K = bulk modulus
M_o = midspan moment due to dead load
V = ultrasonic wave velocity
Y = bulk density
δ = midspan deflection per unit load
ν = Poisson's ratio
ρ = density

Instantaneous deflection

The results permitted comprehensive analysis of the influence of the sand concentration on the elastic moduli.

Young's modulus—From the slopes of the load-deflection curves, (Fig. 3), mean values of Young's modulus were calculated for 21 different sand concentrations using the equation of an elastic beam

$$E = \frac{l^3}{48\delta I} \quad \text{.....(1)}$$

The average of the above values, plotted against the sand concentration (Fig. 7), show a behavior similar to the previous curves, namely an almost linear increase up to a maximum at about 52 percent, and a steep decrease above it.

The empirical equation obtained for the linear segment in Fig. 7 is

$$E_m = E_p\,(1+1.5\,c_v) \quad \text{.....(2)}$$

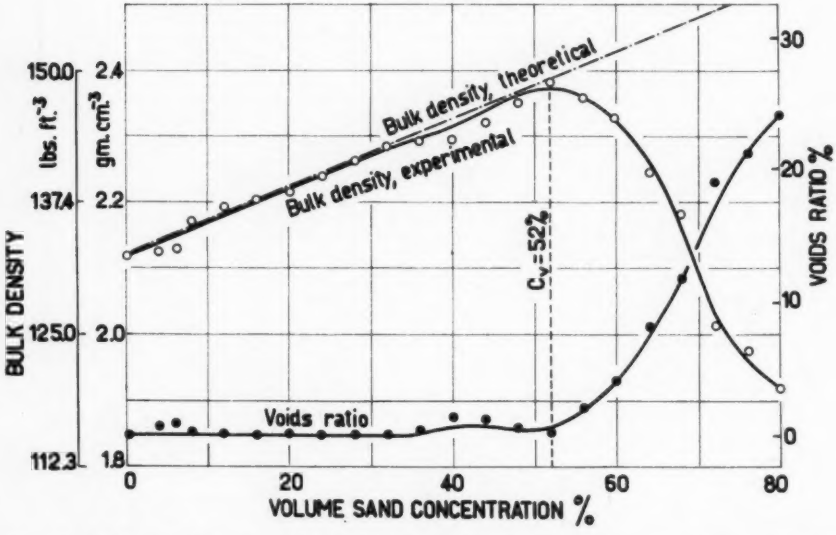

Fig. 5—Bulk density and void ratio of mortar beams versus sand concentrations

This result is similar to the theoretical formula obtained by Hashin[4] for rigid spheres dispersed in an elastic medium. Substituting in the values obtained in this work for Poisson's ratio, we obtain a result not much different from the experimental formula obtained for mortar in the present work:

$$E_m = E_p\,(1+2\,c_v) \qquad (3)$$

Poisson's ratio—Given Young's modulus, the bulk density and the ultrasonic wave velocity for a given beam, Poisson's ratio could be calculated with the inter-relationship equation for an elastic material transmitting waves. In this equation, E is the dynamic modulus. In the present study, static tests were made. However, since time factor was here eliminated, and E was measured at low stresses, the author could see no objection to its use in the dynamic equation.

$$V^2 = \frac{E\,(1-v)}{(1+v)\,(1-2v)} \qquad (4)$$

The averages of Poisson's ratio calculated as above, are plotted against sand concentration in a curve showing a slow decrease up to 50 per-

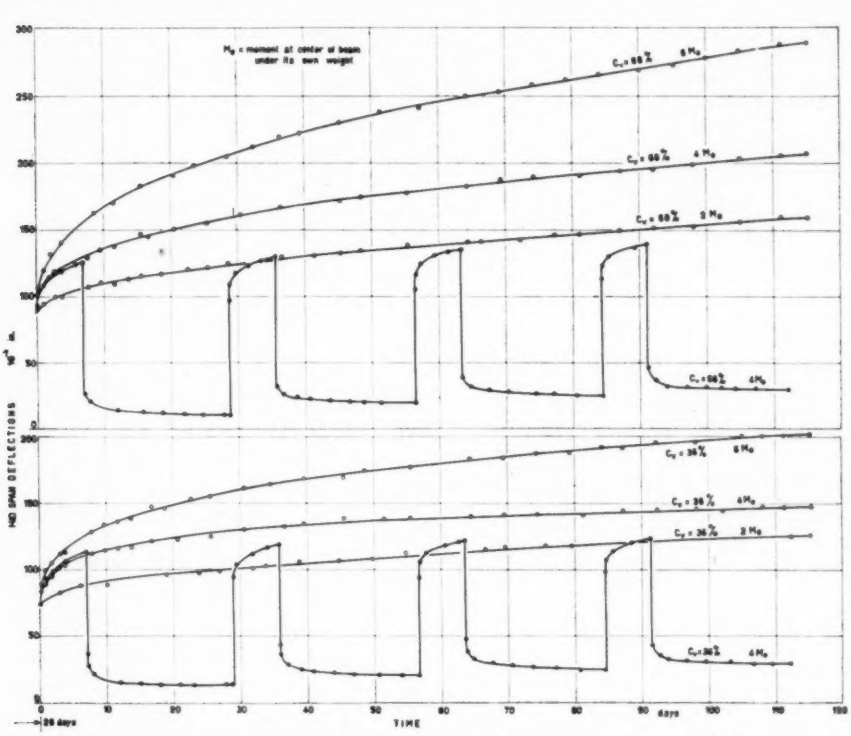

Fig. 6—Instantaneous deflections and creep of mortar beams in flexure under continuous and periodic loading

TABLE 1 — PERIODIC LOADING CREEP TEST DEFLECTIONS

Sand volume concentration, c_v	36 percent				68 percent			
Cycle No.	1	2	3	4	1	2	3	4
Instantaneous deflection (1), 10^{-4} in.	74	73	73	73	90	85	85	87
Total deflection after 7 days (2), 10^{-4} in.	112	117	119	120	127	131	135	138
Nonreversible creep (3), 10^{-4} in.	12	19	23	27	13	21	26	29
(4) = (2) −[(1)+(3)] Reversible creep, 10^{-4} in.	26	25	23	20	24	25	24	22

cent, beyond which statistical scatter precludes the determination of its exact character (Fig. 7).

Modulus of rigidity and bulk modulus—Given E and ν, the modulus of rigidity G and the bulk modulus K could be calculated from Eq. (5) and (6). Their means are similarly plotted against the sand concentration (Fig. 7) and the curves obtained again show a slow and almost linear increase up to a maximum of 50 percent, and a steep decrease above it.

$$G = \frac{E}{2(1+\nu)} \quad \text{(5)}$$

$$K = \frac{E}{3(1-2\nu)} \quad \text{(6)}$$

Permanent set—The load-permanent set curves (Fig. 4) show that below $c_v = 50$ percent, the permanent set is extremely small (about 1-2 percent of the elastic deformation), and can be neglected in view of the local residual deflections resulting from the stress concentrations at the points of support. It can thus be said that brittle mortar has no permanent set.

On the other hand, at sand concentrations above 50 percent, the permanent set increases progressively and exceeds 10 percent of the elastic deflection above a certain stress level (on the average, 60 percent of the ultimate load), while below that level it can be neglected as before.

Time-dependent deflection

The creep values from Fig. 6 plotted against the applied moments over 50 and 100 days, are fairly straight lines (Fig. 8). This indicates proportionality between creep and stress, as shown in earlier studies by Glanville,[5] Reiner,[6] and Freudenthal.[7] This proportionality justifies plotting the creep against unit stress.

According to Neville[8] and others, there is a clear correlation between actual-breaking stress ratio and the creep of different concretes at a given actual stress, whereas a comparison between the creep of different concretes at the same stress has no comparative significance in view of the difference in strength. Hence, a comparative study of the time behavior of different mortars should be based on creep curves corresponding to the same actual-ultimate stress ratio, as was in fact done for the three following concentrations, 0 percent, 36 percent, and 68 percent (Fig. 9). The curve for $c_v = 0$ percent was taken from a study by Glucklich and the author[9] on hardened cement paste beams identical with those used in the present study.

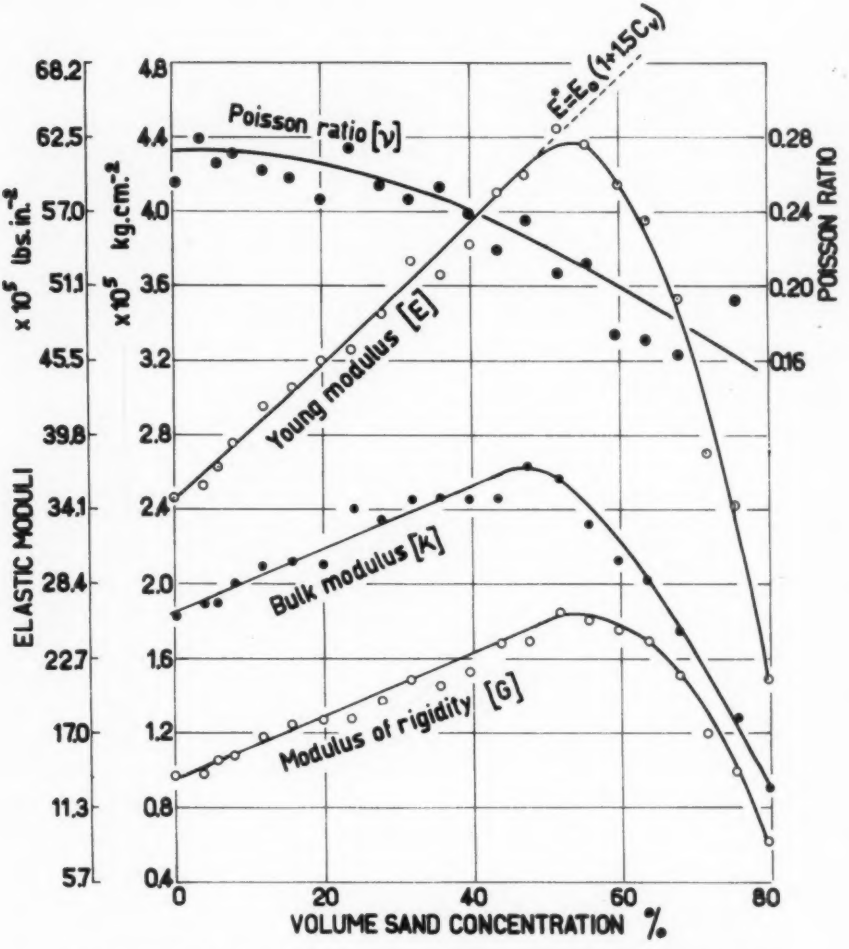

Fig. 7—Elastic moduli of mortar beams versus sand concentrations

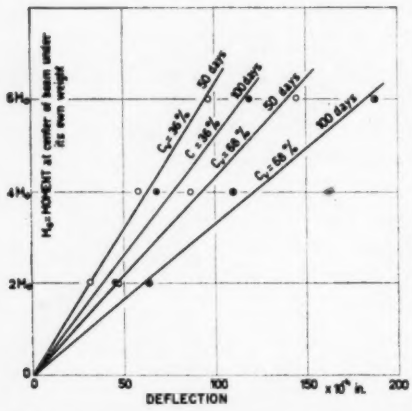

Fig. 8 — Load versus creep in mortar beams over 50 and 100 days under load

Owing to the lack of information in the present study on the creep behavior at other concentrations, a complete picture of the influence of the sand content could not be obtained, but Fig. 9 at least shows the following trend; an increase in sand concentration from 0 to 36 percent reduces creep to about one half, while further increase reduces it still further, but at a much slower rate. This statement is subject to the following qualification: when c_v increases from 36 percent to 68 percent both sand content and air voids increase, the former tending to reduce the creep and the latter to increase it. To permit comparison the effect of air voids was eliminated here by relating stress to strength.

CONCLUSIONS

Mortar consists of a continuous cement mass in which comparatively rigid particles are dispersed. Obviously, a change in the concentration of these particles would lead to a quantitative change in the properties of the material. Since at sand concentrations below 50 percent the voids ratio is almost zero (Fig. 5) the mortar at this stage comprises two phases only, hardened cement paste and sand. An increase in the proportion of the sand, which is denser and more rigid than the cement paste, would lead to a corresponding increase in the density and rigidity of the material. Fig. 2, 5, and 7 actually show a slow, almost linear, increase in these values up to $c_v = 50$ percent.

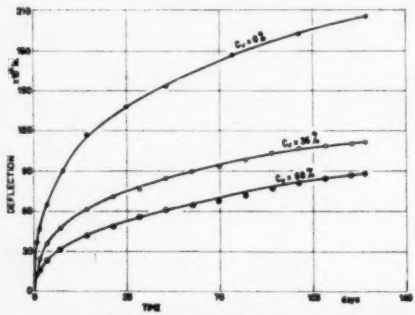

Fig. 9—Creep of mortar beams in flexure proportionately reduced to a load of 25 percent of breaking load

In view of the continued linearity, up to failure, of the stress-strain curves (Fig. 3 and 4) and of the type of failure characteristic of this range of concentrations, low-sand mortar may be regarded as a brittle material, similar in behavior to the hardened cement paste which is its main constituent, while the sand particles are dispersed in it without mutual contact and without qualitative influence on its behavior.

This situation undergoes a drastic change at a certain point which is within the range of 50 percent to 60 percent. Particles are then brought into contact, thus introducing intergranular friction as a factor affecting the behavior of the mortar. In addition, as can be seen from Fig. 5, the increasing voids ratio indicates the introduction of a third phase into the material, namely, air voids. Both these new factors produce a qualitative change in the properties of the material. The deviation from linearity of the stress-strain relationship at high sand concentrations indicates a structural change in the material. This change, however, does not result in immediate failure owing to the ability of the material to transmit stresses by friction. The void ratio, as can be seen from Fig. 10, is the cause of the steep decrease in rigidity, density, and strength at high sand concentrations. The dominant phase in this material are the sand particles, bonded together by friction and a cement filling providing a low-strength coating. Friction and high voids ratio are the cause of the nonbrittle character of mortars with sand contents above 60 percent.

In the absence of adequate experimental data, no far reaching conclusions could be drawn with regard to the influence of the sand concentration on the parameters of the *time-dependent* deflections except for the evident trend towards a decreased deformability with increasing concentration. Under these circumstances, the rheological model of the two types of mortar is not mentioned at this stage. The author hopes, however, that further studies now in progress will provide a more comprehensive picture.

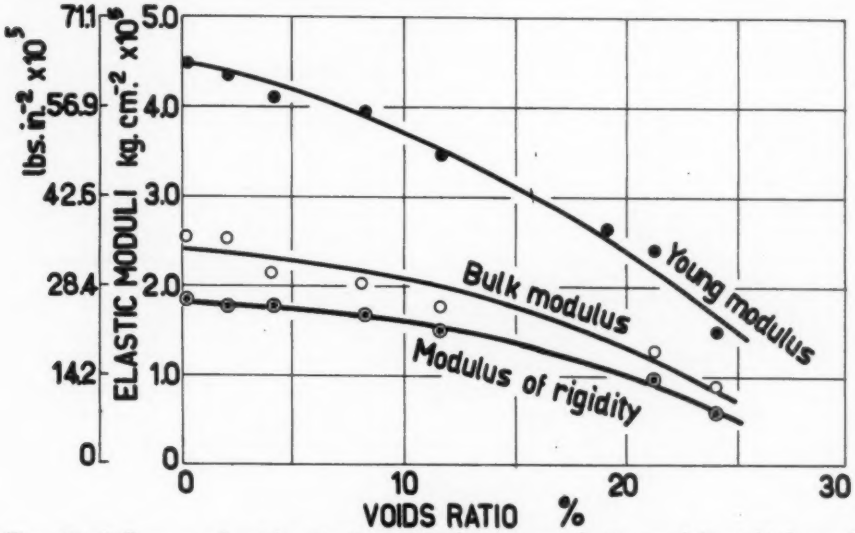

Fig. 10—Influence of void ratio in mortar beams on elastic moduli at high sand concentrations

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Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Influencia de la Concentración de la Arena en las Deformaciones de Vigas de Mortero Sujetas a Esfuerzos Ligeros

Se sometieron vigas delgadas de mortero, preparadas con diferente concentración de arena, a una serie de pruebas consistentes en cargarlas y descargarlas instantáneamente, arrastres prolongados y pruebas de resistencia y densidad para establecer la influencia del volumen de concentración de la arena en el parámetro reológico del mortero.

Los resultados obtenidos demuestran que hasta una concentración de un volumen del perciento, la rigidez, densidad y resistencia aumentan ligeramente

y casi en línea proporcional con la concentración de la arena. Además, la relación entre los esfuerzos y deformaciones en este grupo es también en línea proporcional hasta el límite de fallo.

Por otra parte, cuando la concentración de arena era más del 60 perciento se observaron puntos de disminuición de las propiedades arriba mencionadas con aumento de la concentración, así como desviación de la línea proporcional de la curva de deformaciones y esfuerzos, aproximadamente al 60 porciento del esfuerzo máximo. La mayoría de los valores llegaron al máximo en el intermedio de fluctuación entre el 50 y el 60 porciento.

L'influence de Concentrations de Sable sur les Déformations de Poutres en Mortier Sous des Contraintes Faibles

En jeu de poutres minces en mortier, préparées avec des concentrations variables de sable, a été soumis à une série d'essais de flexion comprenant des chargements et déchargements instantanés, à des essais de fluage à longue durée et à des essais de résistance et de densité, pour établir l'influence de la concentration par volume de sable sur les paramètres rhéologiques du mortier.

Les résultats démontrent que jusqu'à une concentration par volume de 50 pourcent, la rigidité, la densité et la résistance augmentent légèrement et presque linéairement avec la concentration de sable. De plus, le rapport contrainte-déformation est également linéaire jusqu'à la rupture dans cette gamme.

Avec des concentrations de sable supérieures de 60 pourcent, cependant, on observe un décroissement rapide pour lesdites propriétés, lorsque la concentration augmente, aussi bien qu'une déviation de la linéairité de la part de la courbe contrainte-déformation á 60 pourcent environ de la déformation limite. La plupart des valeurs atteignent un maximum dans la gamme intermédiaire de 50-60 pourcent.

Einfluss der Sandkonzentration auf die Verwerfungen von Mörtelträgern unter niedrigen Beanspruchungen

Mit verschiedenen Sandkonzentrationen angefertigte schlanke Mörtelträger wurden einer Anzahl von Biegungsprüfungen, die aus Augenblicksbe- und -entlastungen bestanden, Standfestigkeitsprüfungen und Festigkeits- und prüfungen unterworfen, um den Einfluss der Sandvolumenkonzentration auf die Formveränderungskonstante des Mörtels festzustellen.

Die Ergebnisse zeigen, dass bis zu einer Volumenkonzentration von 50 prozent Steifigkeit, Dichte und Festigkeit geringfügig und fast linear mit der Sandkonzentration zunebmen. Ausserdem ist die Spannungs-Dehnungsbeziebung in diesem Bereich bis zum Bruch ebenfalls linear.

Andererseits wurde bei Sandkonentrationen über 60 Prozent sowohl ein steiler Abfall der obigen Eigenschaften mit zunehmender Konzentration als auch eine Abweichung von der Linearitat der Spannungs-Dehnungskurve bei ungefähr 60 Prozent der Bruchfestigkeit festgestellt. Die meisten Werte erreichen ein Maximum in dem Zwischenbereich von 50 bis 60 Prozent.

Title No. 58-30

Full Scale Testing Develops Efficient Preloaded Concrete Pillars

By JOHN J. REED and C. DAVID MANN

Describes an effort to develop stronger, more effective concrete pillars for underground mine support. Experience indicated that pillars that were not preloaded do not carry appreciable loading until roof rock and adjacent rock pillars have begun to fail. An analysis is presented of data collected during construction, testing, and preloading of 10 large prestressed concrete mine pillars.

■ THE WORK DESCRIBED IN THIS PAPER has been a joint effort of the Mine Research Department and the Mine Operating Department of the St. Joseph Lead Co. to develop stronger more effective concrete pillars for underground mine support.[1] During the past 30 years, concrete pillars up to 12 ft in diameter by 50 ft high have been built occasionally where needed in the Southeast Missouri lead mines. In at least two instances, such pillars were built with the object of 100 percent recovery of high grade ore pillars, but generally, and certainly with present costs, these pillars are used only to safeguard important installations. Past experience, substantiated by 20 years of compression measurements on pillars built in 1938, has demonstrated that concrete pillars which are not preloaded will never carry appreciable weight by natural loading until the roof rock and adjacent rock pillars are already failing. By that time it is too late; the roof caves away from the pillar, and it stands visible for a time in the rockpile, intact, but useless. In early attempts at preloading, some 16 screw jacks were placed atop the pillars and tightened against the roof.[2] However, their total force was only of the same order as the weight of the pillar itself, about 80 psi on the pillar area, and a much more powerful preloading force was clearly required. Thesis work in mining at the University of California,[3] and subsequent knowledge of the use of flat jacks in France, led to the solution described in this paper.

PILLAR CONSTRUCTION

Until recently, all pillars were built entirely of cast-in-place concrete, and heavily reinforced with ordinary internal reinforcing bars. Sizes ranged from 8 to 12 ft in diameter and from 20 to 50 ft in height. Since 1955, through considerations of support distribution, and to provide a pillar of a more convenient size, a 6 ft diameter has been adopted. A 35-ft pillar is the highest of this size built to date. In a further effort at convenience and cost cutting, precast rings have been tried. These are 1 ft high, with a 2 ft hole through the center, and a 3-in. step on top and in the bottom so that they key together as stacked. With existing equipment, these rings can be stacked within about 4 ft of the roof, after which the core and top portion are filled with concrete mixed on the site. The final 6 to 8 in. is later tamped full of dry-pack grout, Fig. 1. These rings contain only light mesh reinforcing to permit their handling during transit and stacking.

This paper describes the construction of 10 pillars. The first seven were built of precast rings, while the last three were cast in place in daily lifts of 4 ft. The concrete in the precast rings was fairly well controlled at a specified 3000 psi strength. Quality control on site-mixed concrete was introduced after building the fourth pillar, and raised the over-all strength of concrete obtained from 500 psi to over 6000 psi. The precast rings have rough upper surfaces, and therefore fit together poorly. The first four pillars were stacked with nothing between the rings, and the joints in the fourth were grouted after stacking. The joints in pillars 5, 6, and 7 were mortared during stacking. In all of these pillars, several rings failed by flexure under preloading, and it was finally concluded that there was no practical way to develop the full potential strength of a pillar built of precast rings.

Fig. 1 — Adjustable form used to contain dry pack grout at top of pillars

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C. David Mann is a mine research engineer for St. Joseph Lead Co. He is involved primarily with rock mechanics instrumentation and ground control.

Thereafter, pillars 8, 9, and 10 were built of high quality concrete placed on site, with no interior reinforcing, but externally prestressed with 1 in. round mild steel bands on 6-in. centers. Anchorage for these half-circle bands was provided by two pairs of heavy 3-in. angles welded back to back. These angles were drilled on 3-in. centers, and placed on opposite sides of the pillar, Fig. 2. Eventually, ⅞ in. high strength bands could be used with standard tank lugs. The bands are tightened to refusal with a heavy impact wrench.

PILLAR PRELOADING

Preloading is accomplished with a system of six triangular flat jacks imbedded in the base of the pillars as shown in Fig. 3 and 4. The manufacture and use of these jacks has been highly developed in France by the Freyssinet Co., represented in the United States by Intercontinental Equipment Co., Inc., of New York. Grids of ½ in. reinforcing bars on 3-in. centers, shown in Fig. 5, are placed 2 in. above and below the jacks for stress distribution. Counter sinks for the two fittings on each jack are provided by covering them with heavy rubber washers, Fig. 4, which can be pried out later. The total area of a 6-ft pillar is 4070 sq in., and the effective area of the six jacks is 2481 sq in., or about 61 percent of the total.

When the pillar is cured and prestressed by banding, it is prepared for preloading. Certain instrumentation is essential during this procedure to control and evaluate the loading. In addition to an accurate hydraulic pressure gage on the pumps, three external measurements are used. Six dial gage extensometers are attached to the

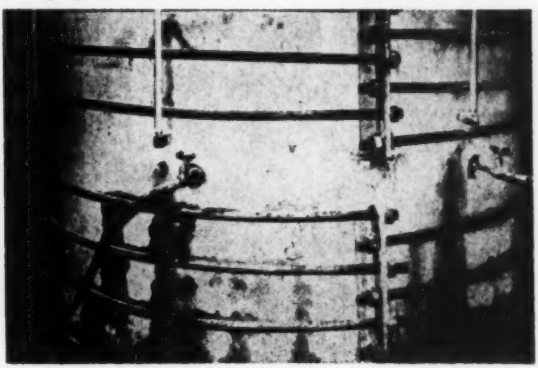

Fig. 2 — Prestressed pillar banding with anchorage of double angle iron. Measuring points straddle incipient crack at level of jacks

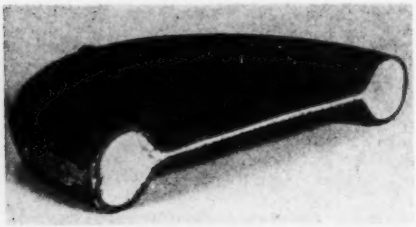

Fig. 3—Cross section of flat jack filled with grout

pillar, one above the center of each jack as shown in Fig. 6. These measurements give an accurate unit compression within the pillar, and warn of eccentric loading. From these data, the Young's modulus is derived. A second set of points straddles the position of the jacks (see Fig. 2) and permits the measurement of the gap, or jack opening, with an inside micrometer as shown in Fig. 7. This measurement is used to calculate the apparent Young's modulus of the entire pillar. It includes not only the effect of any slack within the pillar, but also the effect of any openings in the rock layers of the roof or floor. A third set of measurements is made with a movable dial gage extensometer between sets of permanent convergence stations in the roof and floor of the mine opening, or stope.[4] These measurements detect local stope divergence during preloading due to closure of open roof and floor layers, or elastic deformation, and perhaps subsequent long term reconvergence due to general stope closure.

Actual loading of the pillars is done with water by manifolding the six jacks to hand operated boiler test pumps, Fig. 8. Generally, a good tight pillar will pick up the load evenly, but sometimes one or more

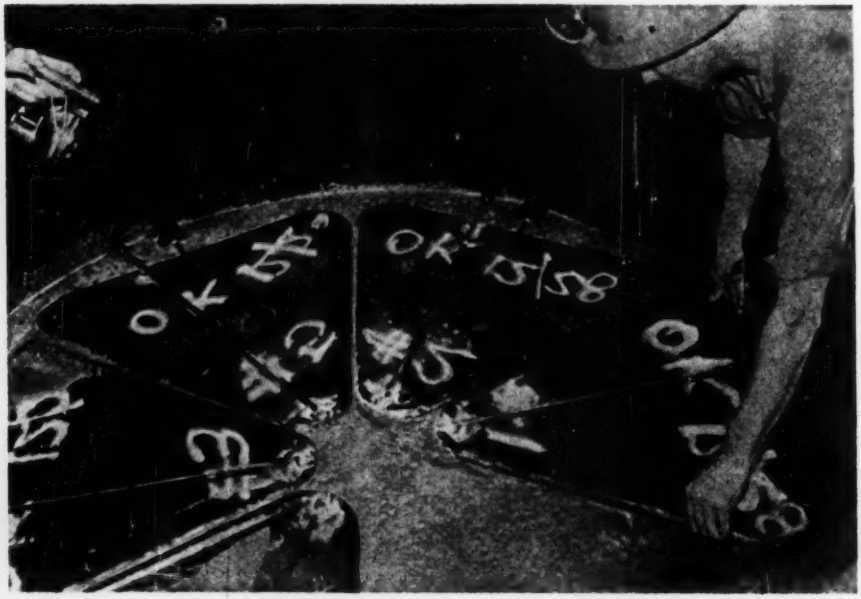

Fig. 4—Positioning flat jacks in base of pillars

of the jacks take most of the load as detected by the extensometers. When this occurs, these jacks are cut out of the system periodically in an attempt to overinflate the balance of the jacks and avoid excessive eccentricity in the loading. The manipulation of the jack pressures is done through a console table (Fig. 8) equipped with a blackboard to record pressures and corresponding extensometer readings. When the desired preload condition is attained with water pressure, the next step is to replace the water with grout, one jack at a time, to render the condition permanent. While one jack is being grouted, the other five hold the load.

A small Moyno pump handles the grout at pressures up to about 100 psi, and high final pressures, with low volumes involved, are produced with a transfer cylinder, indicated in the schematic diagram, Fig. 9. This cylinder has a 2-in. bore and 30-in stroke. A free piston separates water supplied by the test-pumps from grout supplied by the Moyno pump. The position of the piston in the transfer cylinder is indicated by the water level in a vertical plastic standpipe shown feeding the test pumps in Fig. 8. A simple plug valve separates the cylinder from the Moyno pump under high pressures, and a special grout valve makes the connection to the flat jacks. This grout valve was invented especially for the job, and is a key factor in the successful application of the system. It makes possible the absolutely leakproof seal of the flat jacks with a standard pipe plug under conditions of extreme pressure. The only expendable parts are a simple coupling on the jack plus the pipe plug. The step by step procedure for grouting the jacks is:

1. Vent pipe, grout valve, and plug valve open
2. Test pump bypass and Moyno bypass closed

Fig. 5 — Reinforcing grids placed 2 in. above and below flat jacks

3. Transfer cylinder piston near grout end
4. Pump grout with Moyno until all water is flushed from jack and thick grout exits from vent
5. Open Moyno bypass, plug vent pipe, open test pump bypass, close Moyno bypass, fill transfer cylinder with grout, (water level rises in standpipe) close plug valve, open Moyno bypass and flush Moyno
6. Close test pump bypass and pressurize jack to final pressure with test pump (water level falls in standpipe)
7. Close grout valve
8. Open grout valve flush plug and plug valve
9. Flush system with water through Moyno pump
10. Disconnect grout valve

When all six jacks have been grouted, there remains an open crack, visible in Fig. 7, which covers 39 percent of the pillar area between and around the jacks. This crack is caulked with mortar or oakum and wedges, and subsequently grouted solid. For long term permanence, the finished pillar should be gunited to prevent the banding from rusting.

RESULTS

The results obtained in building and preloading the ten pillars discussed are summarized in Table 1. As will be shown later, to fulfill its purpose, an artificial mine pillar must be: (1) preloaded to a high proportion of its ultimate strength, (2) as strong as economically possible, and (3) highly resistant to shortening, i.e., have a high Young's modulus.

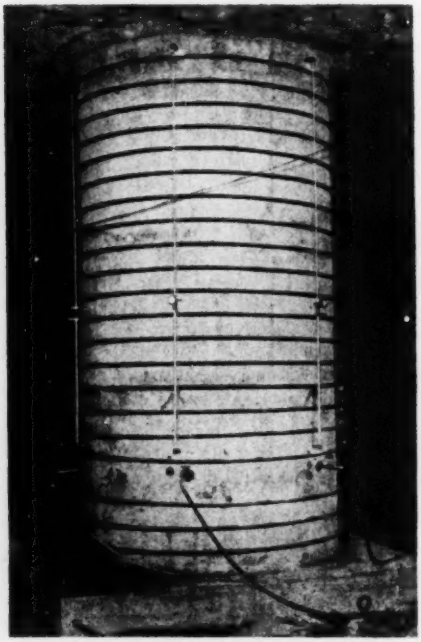

Fig. 6—Completed pillar, prestressed and instrumented for preloading

The superiority of Pillars 8-10 in meeting these requirements is strikingly apparent from the tabulation. By testing full size pillars to failure, this work has discovered and emphasized the weaknesses, inherent and otherwise, in the pillars being built. This is certainly of the utmost importance in trying to build truly strong effective pillars as cheaply as possible.

Rupture seems to be associated with excessive jack openings as well as high pressures (Table 1). Theoretically, these jacks have a "run" of 1 in., and actually, one jack in Pillar 3 opened ¾ in.

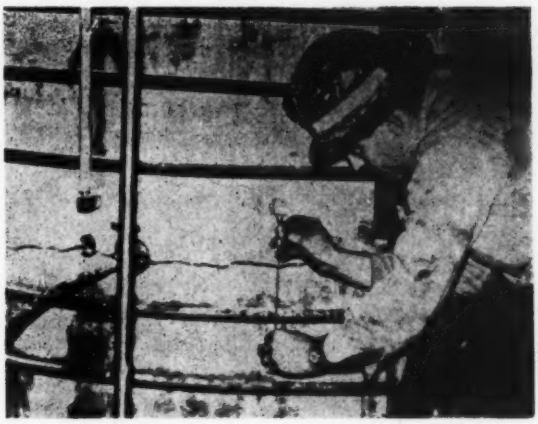

Fig. 7 — With preloading well advanced, inside micrometer is used to measure jack opening

Jacks have been repeatedly proof tested in the laboratory at about 7000 psi, supported only over their flat central portion. Under these conditions, the edge weld is under uniform tension. Although unsubstantiated to date, it is postulated that, in contrast, a jack imbedded in concrete has its outer roll partially supported during inflation, and the edge weld is subject to additional flexure stresses as it tries to push out into the open crack. These flexure stresses added to the uniform tension probably cause local concentrations sufficient to ini-

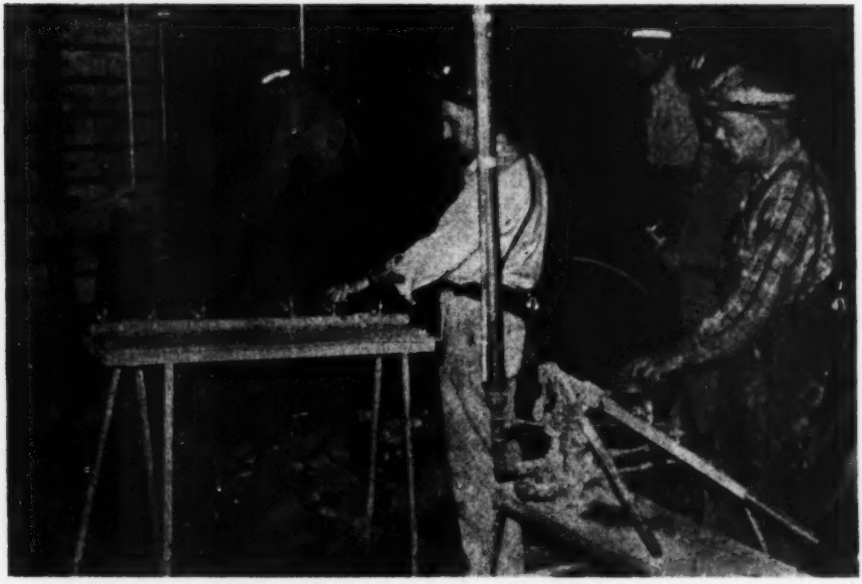

Fig. 8—Boiler test pumps used for preloading. Manifold control console on the left, and grout mixer in foreground

tiate failure, and the weld rips open. Whatever the mechanics of these ruptures may be, where excessive dilation is expected, the simplest solution appears to be the use of two sets of flat jacks in the pillar, one above the other. In this way, the first set of jacks pressurized would give only moderate preloading, but would take up nearly all the slack and creep in pillar, roof, and floor. The second set of jacks should then attain a high preload with little dilation.

STRUCTURAL CONSIDERATIONS

A study of the properties of the rock and concrete pillars involved in the area described in this paper will emphasize the importance of preloading any structure used for permanent underground support. These considerations do not necessarily apply to such temporary yielding supports as timber or yieldable props or arches. They are based on actual conditions and measurements in the area. The rock properties are: $E = 5.5 \times 10^6$ psi; compressive strength = 30,000 psi; overburden = 475 ft; and weight = 182 lb per cu ft. Therefore, the pressure = 1.26 psi per ft of depth. The normal original vertical stress at mine level = 475 ft $\times$ 1.26 psi per ft = 600 psi. At 90 percent extraction, in a stope of semi-infinite extent, the normal pillar stress should become: $600 \times 10 = 6000$ psi. The properties of the concrete were: $E = 5.5 \times 10^6$ psi and compressive strength = 5000 psi.

Pillars 2 through 7 have an average height of 248 in. In preloading these pillars, an average jack opening of 0.284 in. was required to develop an average load of 988 psi in the concrete pillars. If the pillars had not been preloaded, to develop this loading naturally would require the adjoining rock pillars to be shortened an equal amount. The fundamental relationship between stress, modulus, and strain is: stress = modulus $\times$ strain. In this case, the additional load necessary to shorten the rock pillars by 0.284 in. would be $5.5 \times 10^6 \times (0.284/248) = 6300$ psi. Therefore, by the time natural ground movement developed only 988 psi in the concrete pillars, the stress on the rock pillars would have to increase from 6000 to 12,300 psi, or 105 percent. This demonstrates the effects of combining the actual conditions of excessive slack or openings in the roof layers with slack and/or "softness" (low modulus) in the pillar itself.

From the results of preloading ten pillars, it is apparent that there is always some slack or separation in the roof and floor rocks. However, for the sake of illustration, let us assume there is no slack anywhere and consider only the relative moduli of the concrete and rock pillars. The modulus, E, measured between points on concrete Pillars 4 through 7, averaged 2,480,000 psi. When the stress in such pillars reaches 1000 psi, the strain = (stress/modulus) = $(1000/2{,}480{,}000) = 403 \times 10^{-6}$ in. per in. If this strain is caused by natural loading, then it must also occur in the adjacent rock pillars, which

means raising their stress by $5.5 \times 10^6 \times 403 \times 10^{-6} = 2220$ psi. to a total of 8220 psi. This emphasizes the importance of building "hard" (high modulus) pillars, even if there is no slack in the roof structure.

Actually, Pillar 8 was an excellent pillar with a modulus of 5.57×10^6 psi, but because of openings in the roof, the flat jacks had to open 0.286 in. before the pillar carried its final load of 2020 psi. Therefore, under natural loading, the adjacent rock pillars would have had to take an additional unit strain of $(0.286/99) = 0.00289$ in. per in. to cause the concrete pillar to carry 2020 psi. This means the stress in the rock pillars would increase by $5.5 \times 10^6 \times 2890 \times 10^{-6} = 15{,}900$ psi, and their total stress would increase from 6000 to 21,900 psi or 265 percent.

Because of separations in the roof layers, Pillar 8 had to be raised almost ⅛ in. before it carried any appreciable load at all. Of the total jack opening of 0.286 in. at a concrete stress of 2020 psi, an opening of 0.227 in. existed at a stress of only 1080 psi. To have reached even this low load level without preloading, the rock pillars would psi, and their total stress would increase from 6000 to 21,900 psi or crease of 210 percent.

From these calculations, based on actual measurements, it is clear that the stresses on the rock pillars in the area studied would have had to increase by 105 to 265 percent before the concrete pillars could carry their present loads, if the latter had not been preloaded. Obviously, a rock pillar already beginning to fail cannot be expected to carry such increased loads without collapsing completely. On the contrary, the effect of time alone will cause its ultimate failure under

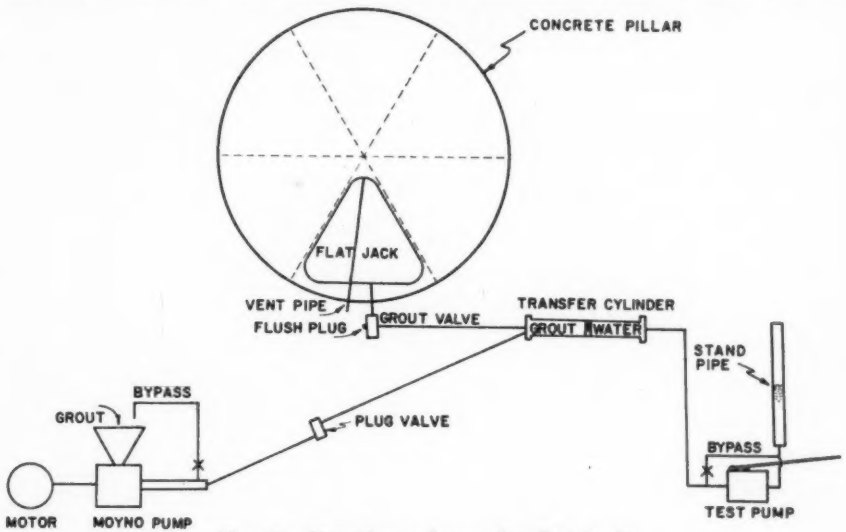

Fig. 9—Grouting scheme for flat jacks

a constant load.[4] Furthermore, stope convergence of this magnitude will generally cause roof failures which will render the area unworkable.

All of these considerations demonstrate that whereas a high modulus pillar is essential, so also is a method for preloading the pillar. Both requirements must be met under actual practical conditions if effective support is to be attained.

To distribute the concentrated load on the roof directly over a preloaded pillar, and to prevent possible local spalling, it is advisable to bolt the roof over and around the pillar location in a close pattern. When the area is completely bolted, the bolts should be retightened repeatedly to eliminate all possible slack in the roof before the pillar is completed. Any slack eliminated in this way will leave that much more "run" in the flat jacks available for actual preloading. The roof was bolted around Pillars 8-10, but the bolts were only tightened once as installed. After preloading these pillars, many of the bolts were obviously loose, and all had to be retightened.

No final conclusion has been reached as to how much preload should be placed on pillars such as those described. Studies by the U. S. Bureau of Reclamation[5] indicate that concrete will support about 70 percent of its ultimate load for an indefinite time. They also indicate that both 18 x 36-in. and 36 x 72-in. cylinders will develop about 85 percent of the compressive strength of the 6 x 12-in. cylinders of the same concrete. In addition to these factors, the coefficient of variation of the compressive strengths of the 6 x 12-in. cylinders should be considered. This coefficient increases as concrete

TABLE 1 — SUMMARY OF RESULTS ON

Pillar No.*	Height, base to roof	Avg grout press, psi	Total load, tons	Avg concrete stress, psi	Avg strength of concrete poured on site, psi†	Avg strength of dry pack top, psi	Avg jack opening, in.
1	16 ft 6 in.	700	840	425	—	3542	0.049
2	14 ft 0 in.	1080	1300	660	507	4996	0.227
3	12 ft 9 in.	1250	1500	765	778	4960	0.366
4	17 ft 5 in.	2400	2880	1470	566	—	0.262
5	28 ft 0 in.	1600	1920	980	5164	—	0.280
6	27 ft 0 in.	1600	1990	980	4764	4140	0.309
7	27 ft 7 in.	1750	2180	1070	3710	3180	0.263
8	8 ft 3 in.	3300	4100	2020	5108	4170	0.286
9	14 ft 2 in.	3666	4550	2240	5177	4770	0.281
10	11 ft 6 in.	3150	3910	1920	6417+	3920	0.332

*1-7, precast rings; 8-10, monolithic site mixed, concrete; 1-4, placed pneumatically, slump perhaps 10 in.; 5-10, concrete hoisted and shoveled into place, slump 2-4 in., mix = 1:1.75:2.65, maximum aggregate 1 in., corresponds to 7.3 bag per cu yd; 1-7, from 2 to 8 rings failed by flexure and crushing; 8-10, No signs of failure, loading discontinued due to limitations of preloading system.

quality control decreases. It is equal to the standard deviation of the test cylinders divided by their average strengths. The standard deviation of the cylinders from Pillar 10 is 892 psi. Therefore, the coefficient of variation for this pillar is (892/6417) $\times$ 100 = 13.9 percent. The criteria generally accepted by the Bureau require that the strength of 80 percent of the test cylinders exceed the design strength. Using this convention with the Bureau's cumulative probability curves, and a coefficient of variation of 13.9 percent, one finds that the design strength should be 88 percent of the average cylinder strength to assure that 80 percent of the cylinders test above the design strength.

If all the factors developed above are now applied to the average compressive strength of concrete in Pillar 10, the result should be the safe permanent load this pillar can carry without reinforcing, and with no factor of safety. Thus 70 percent $\times$ 85 percent $\times$ 88 percent = 52.5 percent, therefore 6417 psi $\times$ 0.525 = 3360 psi.

The design philosophy for underground structures should of necessity be different from that employed on the usual civil engineering structure. It is obvious from theoretical considerations, and has been amply demonstrated by precise measurements, that no mine support structure can carry load unless it is either purposely preloaded, or the mine structure deforms in some way to cause load to be transferred to the support structure. It is also obvious that all truly elastic deformation of the mine structure is complete as soon as the opening is made, and before any supports can be installed. Therefore, any subsequent deformation of the mine structure is, in reality,

PRELOADED CONCRETE MINE PILLARS

No. jacks ruptured	Apparent Young's modulus, based on jack opening, *E*, psi	Young's modulus, based on compression within pillar, *E*, psi	Average local stope divergence		
			No. of stations	Average divergence, in.	Avg distance from pillar, ft
0	1,710,000	1,600,000	6	0.0007	2.70
0	489,000	2,170,000	6	0.0191	3.25
1	320,000	552,000	6	0.0167	4.88
0	1,170,000	3,790,000	4	0.0025	12.52
1	1,180,000	2,850,000	§	§	§
0	1,030,000	1,940,000	§	§	§
0	1,350,000	1,340,000	§	§	§
3	700,000	5,570,000	4	0.112	1.24
			4	0.013	4.84
2	1,350,000	5,760,000	4	0.096	1.31
3	798,000	6,260,000	4	0.069	1.22
			4	0.012	4.87

†Based on 28-day tests of 6 x 12-in. cylinders; 2 of 8 cylinders from Pillar 10 exceeded 7070 psi, the capacity of the testing machine.
§Too high for extensometer.

incipient failure, and the structure will get progressively weaker with time. Under these conditions, it is apparent that underground support structures should:

1. Be placed as soon as possible
2. Be preloaded almost to their yield point
3. Be designed with little factor of safety other than their yield range itself.

For Example: 1. A roof bolt with a minimum yield point of about 21,500 lb, and a minimum breaking strength of 32,000 lb, should be placed with an initial tension of about 16,000 lb. Thus it will offer a maximum of reinforcing effect to the undiminished strength of the rock, rather than requiring the rock to commence failing before the bolt works to capacity. So placed, the bolt actually has a safety factor of 2 provided by its yield range. In contrast, if the bolt is placed at a low initial tension, and the rock begins to fail, there is no reason to believe it will ever stop failing, and the bolt will eventually fail also in spite of its high factor of safety. Stated another way, the factor of real importance is the safety of the entire composite structure, and not of the individual bolt.

2. Prestressed, preloaded concrete pillars should be loaded close to their yield strength, because this will reduce the critical load in the adjacent rock structure and prevent its continued failure. The safety factor of the concrete pillar is the yield range of its prestressed steel banding.

To date, no properly built pillar has been loaded to failure, so the actual reinforcing effect of the prestressed bands is not known. It is believed that by placing two sets of flat jacks in each pillar, one above the other, the first set can eliminate the slack in the structure, and the second will attain pressures up to 5000 psi, which will amount to pillar stresses of over 3000 psi. If this causes no signs of failure, the amount of prestressed banding might well be reduced, or the preload increased. Research by John L. Burdick at M.I.T.[6] has shown that prestressed spiral reinforcing can increase the ultimate strength of a concrete test cylinder as much as eight times. A simple rule of thumb which has been developed by such work is provided by the formula:

$$f_p' = f_c' + 4f_r$$

where f_p' is the ultimate stress on reinforced pillar cylinders, f_c' is the ultimate stress in plain concrete test, and f_r is the radial stress developed by prestressed reinforcing. If it is assumed that the banding described in this paper develops about 500 psi radial stress, which is reasonable, then the ultimate strength of a banded pillar containing 6000 psi concrete should be about 8000 psi. Using the safe permanent load, 3360 psi, previously calculated for Pillar 10, the fin-

ished banded pillar should have a safe permanent capacity of about 5360 psi.

Flat jacks are a convenient, inexpensive means of developing exceedingly large forces. To the knowledge of the authors, their use is almost unknown on this continent, and they are a valuable potential tool in all phases of heavy construction. In Europe and Africa, flat jacks have been used in prestressing dams, bridges, tanks, foundations, building structures, and airport runways as well as for experimental purposes in the field of rock mechanics.

Flat jacks in underground sites offer the possibility of testing full size structures too big or too strong to be tested in any testing machine. For example, the loading of Pillar 9 represents a full scale load test of 9100 kips which is nearly twice the capacity of the largest testing machine in existence.

CONCLUSIONS

Analysis of data collected during the construction, testing, and preloading of ten large prestressed concrete mine pillars leads to the following conclusions:

1. Roof over and around the pillar should be pattern bolted, and bolts should be retightened several times to eliminate all possible slack before building pillar.

2. Pillars should be built monolithically of high grade concrete, and prestressed with external banding.

3. A second set of flat jacks should permit high ultimate loading after the first set eliminates excess structural slack.

4. Pillars should have highest possible modulus of elasticity for maximum effectiveness.

5. Actual stope divergence of appreciable magnitude can be obtained by preloaded pillars. Thus the miner can at last take an active as opposed to a passive role in ground support. The stope conditions can now be improved rather than simply maintained. For the first time, to the knowledge of the authors, there is a controllable force available for ground support which approaches in magnitude the forces which must be withstood.

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Discussion of this paper should reach ACI headquarters in triplicate by Feb. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Pruebas en Escala Completa para la Obtencion de Pilares de Concreto Pre-Comprimido

Se describen los trabajos hechos para la obtención de pilares de hormigón más fuertes para puntales subterraneros de minas. La experiencia ha indicado que los pilares que no fueron pre-cargados no llevaron carga apreciable, hasta que los pilares adyacentes y el techo de roca empezaron a fallar.

Se presenta un análisis de los antecedentes obtenidos durante la construcción, pruebas y pre-cargas de 10 columnas o pilares grandes para mines, construidos de hormigón pretensado.

L'essai en Vraie Grandeur Assure un Pilier en Béton Préchargé Efficace

Décrit un effort de mettre au point un pilier en béton plus résistant et plus efficace pour support dans les mines souterraines. L'expérience a montré que les piliers non préchargés ne portent pas une charge appréciable que jusqu'à ce que la banc de ciel et les massifs voisins commencent à faiblir. Il y a une analyse des données ramassées au cours de la construction, l'essai et le préchargement de 10 grands piliers de mine en béton.

Naturgetreüer Prüfversuch entwickelt leistungsfähige vorbelastete Betonstützen

Beschreibt die Bemühung, starkere, wirksamere Betonstützen für den Untertageausbau in Zechen zu entwickeln. Die Erfahrung zeigte an, dass nicht vorbelastete Stützen keine merkbare Last tragen, bevor nicht das Hangende und die bemachbarten Gesteinssäulen angefangen haben nachzugeben. Eine Analyse der Mess- und Versuchswerte, die während der Konstruktion, dem Prüfen und Vorbelasten von 10 grossen Grubenstützen aus Spannbeton gesammelt wurden, wird aufgeführt.

BIBLIOGRAPHIES

LIMIT DESIGN FOR REDUNDANT REINFORCED CONCRETE STRUCTURES

By M. Z. COHN*

FOREWORD

The achievements in limit design have been less spectacular than those for structural concrete or structural steel. However, in the last decade a marked trend towards using limit design has been developing.

This is easily shown by a glance at the following bibliography, which lists more than 80 percent of the literature on limit design for concrete structures, published during the last 10 years.

The increasing use of limit design may be explained by theoretical as well as by practical reasons. From a theoretical viewpoint limit design reduces the gap between theory and actual phenomena; it allows a more accurate estimate of the carrying capacity of structures and provides a logical approach in the analysis of both the structure and its members on a common basis, i.e., their corresponding ultimate strengths. From a practical viewpoint limit design is, generally, more economical with building materials and leads, where applicable, to simplified design procedures, as compared to conventional, elastic design.

The need for a comprehensive bibliography of limit design for structural concrete is, therefore, much needed at the present time.

In their valuable paper[106] Yu and Hognestad have presented a bibliography of some 60 entries. However, this bibliography is limited because two-thirds of the references refer to the ultimate strength and plastic strength of steel structures. In addition, too, some major European contributions are omitted.

These omissions have led the author to compile the present bibliography, comprising the limit design of redundant reinforced and prestressed concrete structures exclusively. This bibliography deals only

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TABLE 1 — THEORETICAL AND EXPERIMENTAL INVESTIGATIONS

Object of investigation	Bibliography No.	
	Theoretical work	Experimental work
Plastic hinges	17, 23, 25, 40, 53, 67	5, 40, 41, 47, 53, 73, 84, 88
Beams	24, 35, 43, 63, 64, 78, 82, 90	5, 7, 27, 37, 60, 70, 79, 92, 97, 102, 107, 113
Frames	34, 39, 50, 63, 64, 85, 101, 110, 117	10, 13, 19, 48, 61, 83, 95
Arches	38, 74	71, 104, 115
Prestressed structures	64, 69, 76, 90, 99	58, 59, 75, 77, 116

with structures made of bars; limit design of plates and shells as well as the limit analysis of structural elements are not included.*

The bibliography includes, however, some references on the properties of plastic hinges in reinforced concrete because they govern the plastic behavior of redundant structures.

Due emphasis is given to European investigations, including Soviet and Eastern European contributions which are less known in the United States.

Hence, the bibliography is intended to cover the entire literature of limit design for concrete structures and to give an almost complete historical and geographical picture of its development.

The bibliography is chronologically arranged starting from 1917 to 1960. Though not initially conceived to demonstrate the plastic behavior of reinforced concrete, the early studies of Bach and Graf[1,2] are included, because of their historical interest.

The topics in the bibliography may be summarized under three headings: general problems, design methods, and experimental research.

Of some significance, perhaps, is the fact that experimental research covers more than a half of the whole literature.

General problems refer to some surveys,[106,111] discussion of limit design fundamentals,[15,21,22,26,42] and structural safety.[46,57,64,65, etc.]

The main references on theoretical and experimental investigations are listed in Table 1.

Attention is called to the outstanding Soviet work of Gvozdev,[11,20,21] Murashov,[23] Thal,[52,105] Krílov,[102,119] and also to some Rumanian contributions.[29,86,95,107,110,116]

An over-all view of the bibliography on structural concrete shows it is vast although the author estimates that it does not represent more than about 15 percent of the more general field of limit design for redundant structures.

*These two subjects comprise two distinct fields, viz., the yield line theory, and the ultimate strength theory, respectively.

Major advances are due to the large number of investigations carried out so far. However, much work is still needed to bring plastic methods of analysis to concrete designers.

It is hoped that the following bibliography will obviate the need for further research in certain branches of limit design for structural concrete.

The author will remain indebted for additions to the bibliography of old and/or recent references which might have been omitted.

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CONTRIBUTIONS TO THE ACI JOURNAL

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Load test conducted on bridge at 1/8 of Oddanchatram-Dharapuram Road

Research Note 1, Highways Research Station, Madras, India, Apr. 1960, 32 pp.

The bridge is a T-beam-slab structure consisting of three spans of 40 ft each. Each span consisted of four tee beams simply supported at the ends without interconnecting diaphragms. The reinforcement in the tee beams had been joined by gas welding and so it was decided to test load the structure.

The procedures adopted in conducting the test, the method of loading, the effect of temperature on the beams, and the test results are described. Distribution of load in the various members of the bridge is discussed. The test load showed that the bridge was safe for the class of loading for which it was designed.

Progressive bridge span design (in Russian)

G. S. Fisher, V. M. Pankratov, and B. V. Klochkov, *Avtom. Dorogi* (Moscow), V. 23, No. 11, 1960, pp. 15-17

Road Abstracts
V. 28, No. 1, Jan. 1961

Explains design and construction of an experimental, prestressed concrete overpass which has recently been completed on the Moscow Ring Road. The structure has no diaphragm and consists of separate T-beams with the top flanges forming the road-way deck. Each beam is in three sections, each of ⅓ span, prestressed to form a single unit. The beams are held together by lateral reinforcement through precast holes in the flanges of the T-beams. The manufacture and assembly of the beams are described; identical hinged metal formwork is used for each component. Specifications are given for the concrete mix and prestressing design. Tables show the man-hours required for each phase of the work.

Construction

Concrete construction in the Gold Coast

F. S. B. Jacobson, *Reinforced Concrete Review* (London), V. 4, No. 7, Sept. 1957, pp. 449-468

Reviewed by William R. Lorman

Deals mainly with postwar concrete design and construction. Describes and discusses construction factors peculiar to the Gold Coast; these include building materials, earthquake-resistant design, labor, wage rates, and brief description of various ministries that administer construction throughout this area. A portion of the paper is devoted to damages sustained by concrete structures during the last three major earthquakes (1862, 1906, 1939). The discussion includes comments by seven authorities and the author's replies thereto; these feature items such as aggregates available, climate, cement storage problems, prestressed concrete

A part of copyrighted Journal of the American Concrete Institute, *Proceedings* V. 58, No. 5, Nov. 1961. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish address of publications added later.

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telegraph poles, concrete curing practices, and opportunities for consultants in Ghana. No references listed; however, the 10 photographic illustrations offer the reader an excellent idea of the concrete architecture that eventually may be characteristic of this region.

Bearing grilles for concrete tower

Progressive Architecture, V. 42, No. 7, July 1961, pp. 137-141

A high-rise office building in Los Angeles demonstrates the ability of concrete to compete aesthetically and economically with other structures. A structural analysis of the building's sculptured bearing-grille tower, transition and base structures, foundations, and lateral loads—with a statement of costs—is presented.

Helena Valley pumping plant and tunnel

Bureau of Reclamation, Building 53, Denver Federal Center, Denver 25, Colo., 163 pp., $3.10

Records the planning, designing, construction, and initial operation of the Helena Valley Pumping Plant and Tunnel, features of the Helena Valley Unit of the Missouri River Basin Project near Helena, Mont. The unit irrigates 17,600 acres and provides municipal water. The pumping plant contains two 150 sec-ft capacity pumps each driven by a 5000-horsepower turbine. The pump discharge lines converge to a single 6 ft 3 in. diameter pipe which terminates at the inlet portal of the 2.65 mile Helena Valley Tunnel. From the outlet portal of the tunnel the water flows by gravity through the Helena Valley Canal along the south, west, and north sides of the Helena Valley.

The volume is divided into seven chapters. Chapter I describes general planning, investigations, and features of the Helena Valley Unit; Chapter II covers geology of the pumping plant and tunnel areas; Chapters III and IV are devoted to design; and Chapter V describes contract administration phases of major construction. Chapters VI and VII comprehensively discuss the construction of the pumping plant and tunnel.

Use of precast concrete members in construction of a new subway line in Hamburg (Verwendung von Stahlbeton-Fertigteiler beim Bau einer neuen U-Bahnstrecke in Hamburg)

G. Mandel and H. Schmidt, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 10, Oct. 1960, pp. 217-224

Reviewed by Rudolph Sziland

Cross section for the two track lines of the new subway follows the old rectangular type with a center column dividing the two tracks. Considerable savings has been achieved, however, by using statically indeterminate rigid frame sections which have been precast and prestressed. The precast members were lowered into the open ditch during the construction. Where open ditch method could not be used tunneling was utilized using circular cross sections built also of prefabricated elements.

Elementary handbook of concrete house construction

Concrete Association of India, Bombay, 1960, 65 pp., 1 Re

Although the material in this book is intended primarily for use in India much of the information given would be applicable to any area where transportation and erection equipment and procedures are available mainly for only small and relatively light precast concrete units. Brochure deals essentially with the use of small precast units in the construction of low cost houses. It explains in simple language how precast units such as hollow concrete block, floor and roof beams, lintels, sills, can be manufactured at the work site; how they can be erected more or less by hand; and how in conjunction with a little cast-in-place concrete work they can be used to make an all concrete house. Typical designs and simple tables are given.

Concrete construction in the Soviet Union

L. J. Murdock, W. J. Beckwith, D. D. Matthews, and K. M. Wood, *Reinforced Concrete Review* (London), V. 4, No. 6, June 1957, pp. 349-378; discussion pp. 379-382
Reviewed by William R. Lorman

This paper, in effect, is a report of observations made by a four-man delegation (representing the Reinforced Concrete Association) during a visit to the USSR in the summer of 1956. The presentation, which represents about 130 man-hr devoted to actual inspection and conferral on technical subjects, covers the following topics: (a) construction administration, standardization, and programs; (b) building research and exhibitions; (c) production of precast concrete floor units, beams, and columns; (d) progress of structures being erected; (e) post-tensioned concrete; (f) construction of Volga Dam; and (g) miscellaneous comments concerning completed apartments(flats) and public buildings. Cities included in this inspection tour were Moscow, Stalingrad, and Leningrad. The paper contains 14 excellent photographs, two tables of data (dimensions of precast units and related concrete mix proportions), and five schematic layouts. Judging by the discussion (which reflects the questions, answers, and opinions expressed by 14 persons), reviewer would surmise that the Soviet use of hydrophobic cement is rather limited and that the manpower available for producing concrete is quite unlimited. Reviewer recommends that the reader also see "Automation in the Concrete Product Factory" (reviewed in "Current Reviews," ACI Journal, *Proceedings* V. 56, No. 8, Feb. 1960, a paper which includes 14 photographic illustrations of Soviet concrete operations. If the reader desires a comprehensive view of current trends in Soviet concrete construction he should supplement these two *Reinforced Concrete Review* references with: *Concrete and Prestressed Concrete Engineering in the USSR*, May 1958, Portland Cement Association, Chicago, 86 pp. This trilogy of recent contributions, in reviewer's opinion offers the best available English language information regarding the many facets that comprise the topic under review.

Construction Techniques

A review of some developments in concrete practice

A. B. Harman, *Reinforced Concrete Review* (London), V. 4, No. 9, Mar. 1958, pp. 523-539
Reviewed by William R. Lorman

Recent developments presented pertain to mixing, batching, transporting, placing, and vibrating of concrete. Advances in design of formwork and how the vacuum concrete process affects the form costs appear worthy of more consideration than the author devotes to these topics; he touches lightly on the subject of time studies. Nevertheless, his conclusions, regarding the need for training technicians in civil engineering methods, seem applicable equally well throughout America.

The design and construction of extension to British European Airways Engineering Base at London Airport

P. B. Edwards and R. B. Rigg, *The Structural Engineer*, V. 39, No. 1, Jan. 1961, pp. 17-31
Authors' Summary

Describes design and construction of hangers and workshops together with maintenance aprons which form part of the extension to Britich European Airways Corporation Engineering Base at London Airport.

Design and construction of the works was based on the maximum use of precast concrete construction and the development of the project on these lines was carried out by close cooperation between the consulting engineers and the contractor.

Scope of project is described in general detail, particular attention being drawn to unusual construction techniques in the field of precast concrete construction. Results of testts carried out throughout the contract are given and compared with design figures.

Importance of premixing cement for the practice of concrete production (Die Bedeutung der Zementleimvormischung fuer die Praxis der Betonherstellung)

K. Wesche, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 11, Nov. 1960, pp. 252-257
Reviewed by Rudolph Szilard

In conventional concrete mixes three components are used: cement, water, and aggregate. Among the 15 factors influencing the ultimate strength of the concrete cement and water are the most important, using premixing of water and cement not only produces better concrete but the mixing is made economical. The results of the pertinent research are shown in various diagrams.

Cold weather concreting

N. M. Brydon, *Reinforced Concrete Review* (London), V. 5, No. 4, Dec. 1959, pp. 205-215: discussion pp. 215-220
Reviewed by William R. Lorman

This is a practical guide based on the author's cold weather concrete construction experiences throughout 15 winters in the provinces of Ontario and Quebec. The paper begins with a comparison of Canadian winter conditions with those in Britain, describes the types of structures the author was concerned with, and points out the important aspects of the preferred methods of constructional operations under severe winter conditions. These facets include heating the concrete mixtures, methods of maintaining necessary temperatures, and costs of protection. The discussion (by ten individuals) covers features such as specifications, hardening of concrete at −14 F temperature, corrosion of steel reinforcement, mixer operation, air-entraining agents, and calcium chloride admixtures. In the opinion of your reviewer, this paper is written well and to the point, is easy to read, and certainly is interesting; the information found therein serves as a good but brief guide for the engineer confronted with the problem of concrete construction during severe winter weather.

The use of concrete on railways

R. L. McIlmoyle, *Reinforced Concrete Review* (London), V. 4, No. 4, Dec. 1956, pp. 221-250
Reviewed by William R. Lorman

This paper is not concerned with small structures nor the miscellaneous precast concrete items that are common to railways. The author examines the service provided, over long periods, by important reinforced concrete structures. The categories are: (a) bridges, (b) coaling and ashing plants, (c) roofs of locamotive sheds, (d) crane gantries, (e) station roofs, (f) wharves and docks, and (g) relatively large one story buildings similar to warehouses. Some of the difficulties experienced during the first half of this century, and corrective actions attempted, pertain to: (1) spalling of bridges situated near sea coasts; (2) chemical action of locomotive exhaust stack gases on coaling and ashing plants and on roofs of enfine sheds; and (3) spalling of reinforced concrete piles. The paper ends with a brief description of current design practices concerning conventionally reinforced as well as prestressed concrete railway structures. In his conclusion the author lists the basic guides for successful concrete construction in this area of application. The paper provides 19 excellent photographic illustrations of typical installations and two schematics pertaining to structural design (one for a concrete and timber quay). All of the structures described, with the exception of the coaling and ashing plants which have proven to be the least satisfactory application of concrete, have necessitated little maintenance or repairs. The author contends that the defects which have arisen, especially spalling, result from faulty workmanship rather than poor design. He lists five references. The discussion contains pertinent points related to concrete density, abrasional resistance, and cracking, and includes comments pertaining to cover for steel reinforcement. Of the ten discoursers who participate in the discussion, Abeles presents four photographic il-

lustrations in support of his contentions that concrete density is more important than depth of cover surmounting steel reinforcement.

Vibration control in piling and blasting

J. H. A. CROCKETT, *Reinforced Concrete Review* (London), V. 5, No. 2, June 1959, pp. 99-137

Reviewed by WILLIAM R. LORMAN

Despite the interesting treatment of the subject, the *Proceedings* of the Institution of Civil Engineers (London) certainly would have been a more appropriate periodical for presentation of this paper. The arrangement is as follows: characteristics of ground vibrations, structural vibrations, human sensitivity, control of pile vibrations, and control of vibrations caused by blasting. The effects of vibrations on new or green concrete are limited to one page (p. 119). In the discussion, however, the effects of various vibrations on concrete are considered at greater length; bond failures (pp. 132-133) and frequency response (pp. 134-136) perhaps are of primary interest to the specialist in concrete structural design.

Development of method of test for concrete mixer performance

W. O. TYNES and K. L. SAUCIER, *Technical Report* No. 6-562, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Mar. 1961, 32 pp.

AUTHORS' SUMMARY

A literature survey revealed that a reliable test method for determining concrete mixer performance efficiency both in laboratory and field did not exist. This investigation was undertaken to develop a satisfactory one. The investigation was divided into four laboratory phases, in which concrete mixtures containing 1½- and 6-in. aggregates were mixed in a 10-S mixer for various lengths of time to simulate well mixed and poorly mixed concrete, and one field phase, in which mixtures containing 6-in. aggregate were mixed in a standard 2 cu yd capacity mixer to simulate well mixed and poorly mixed concrete. Three samples from each batch were found adequate for testing. Tests found to be the most reliable were: (a) unit weight of air-free mortar, (b) percentage of coarse aggregate in the concrete, (c) water content of fresh mortar, and (d) cement content of dry mortar. Maximum allowable variations were established for each test. The test method developed is included as an appendix.

Note

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Prepacked concrete (Béton de blocage et mortiers activés)

M. J. CHEFDEVILLE, Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 12, No. 144, Dec. 1959

AUTHORS' SUMMARY

After first defining prepacked aggregate concrete, the author describes the different processes used to activate the mortars and give them the qualities necessary for a good flow into the channels and a good injectability into the skeleton of pebbles of the prepackaged aggregate concrete. To appraise these qualities, he enumerates the labortary tests that have made it possible to establish the criteria of injectability. Tests on concrete injected with activated mortars make it possible to judge the mechanical qualities of these concretes in relation to traditional concretes of the same composition and the same batching in relation to their age. All these tests, finally, make it possible to compose prepackaged aggregate concretes in terms of the raw materials available on the site and to check their satisfactory execution.

Dams

Erfenis Dam completed

Public Works of South Africa (Johannesburg), July, 1960, pp. 12-17.
Reviewed by D. G. NORMAN

Probably the first dam in southern Africa constructed entirely by the intrusion grout method. Part of Sand-Vet Irrigation Scheme, south of the Orange Free State gold fields, this gravity section dam has net capacity of 1.39 times the mean annual runoff, and is approximately 100 ft high x 1600 ft long. Provision is made for future raising by post-tensioning to cope with steady silting-up. Paper gives general description of construction.

Colcrete method for the construction of dams and other parts of hydroelectric plants (Das Colcrete-Verfahren beim Bau von Staumauern und anderen Wasserkraftanlagen)

G. BRUX, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 3, Mar. 1960, pp. 91-102
Reviewed by FERDINAND S. ROSTASY

The main features of "colcrete" concrete are the production of a colloidal cement mortar in a high speed mixer and its injection into a structure of coarse aggregates (>1½ in.). The grouting of the aggregates can be done in various ways. The aggregate particles must have sufficient strength and should be round or cubic in shape. Substantial savings of cement are possible; fly ash and pozzolanic materials can be successfully used. Shrinkage is reduced; cooling of colcrete is usually inexpensive. The colcrete method was used in a wide variety of projects: dam constructions, large foundations, locks, piers, reactor, and shields. Other variations of colcrete are colgunite which has the same function as shotcrete and colcrete-injection grout for pressure grouting of joints, prestressing ducts, etc.

Describes the necessary installations and equipment on the site where colcreting is to be performed. Discusses several examples where colcrete was successfully used: Oued Milok and Bimont dams in France, and others. Grouting jobs on tunnels and shafts are described. Concluding it can be said that colcrete will be chiefly competetive in countries where skilled workers are scarce and hand labor is cheap. It is today a fast and sound construction method well established in modern concreting practice.

Design

Shear failure and bending moment (Schubbruch und Biegemoment)

H. BAY, *Beton und Stahlbetonbau* (Berlin), V. 55, N. 10, Oct. 1960, pp. 230-235
Reviewed by RUDOLPH SZILARD

Present shear theories follow mostly the theory introduced by Mörsch. The latest investigations indicate, however, that the ultimate shear load capacity of a reinforcing concrete beam is a function of a shear-moment. The author treats shear theories of simply supported beams, (a) with straight reinforcing bars, (b) with complete shear reinforcing (c) beams having the same factor of safety for shear and moment failures. The importance of adequate bond between concrete and reinforcing bars is emphasized.

Practical design of reinforced concrete columns (Le calcul pratique des poteaux en béton armé)

GEORGES LAVAL, Editions Eyrolles and Gauthier-Villars, Paris, 1959, 126 pp., 16 NF (17.10 NF foreign)
Reviewed by ARON L. MIRSKY

A practical design manual, covering the design of columns in accordance with French specifications B.A. 1945 (a comparison with the 1934 specifications is also given) and time and cost estimates. First part (pp. 15-50) gives theory and examples of column, reinforcement, and tie design; second part (pp. 53-70) covers estimates, while third part gives tables from which sections, including reinforcement, can be selected, with an explanation of their use.

Spiral staircases

D. E. Ryder, *The Structural Engineer*, V. 39, No. 2, Feb. 1961, pp. 67-70

Author lists procedure for determining bending moment of spiral staircases using a graphical method.

European Committee on Concrete report on a meeting at Vienna in April 1959

A. L. L. Baker, *Reinforced Concrete Review* (London), V. 5, No. 4, Dec., 1959, pp. 247-263
Reviewed by William R. Lorman

Presents a summary of ultimate load methods (for reinforced concrete design) considered acceptable by the European Committee for eventual incorporation in the proposed International Code of Practice. Professor Baker's previous report appeared in the *Reinforced Concrete Review*, 1958, V.4, No. 11, pp. 637-677 and is reviewed separately. This new report covers assumptions, theoretical aspects, and official conclusions reached by the Committee relative to the following features: (a) pure bending, combined bending, and compression; (b) effective width of T-beam flanges; (c) calculation of deformations; (d) calculation of crack widths and recommended limits; and (e) safety factor values. For details and corroborative test data the reader must refer to the various bulletins (noted in the report) issued by the committee. The synopses of the various design features enumerated herin are supplemented by 15 diagrammatic figures and three tables; an appendix indicates the 11 divisions comprising the Committee and gives the names and addresses of the respective chairmen. Design engineers who desire correspondence membership in the European Concrete Committee, or who desire copies of the bulletins, will find necessary information at the end of the report. There is no customary discussion by the members of the Reinforced Concrete Association. Professor Baker is the Association's official representative on the European Committee.

Introduction to structural mechanics

Paul Andersen and Gene M. Nordby, The Ronald Press Co., New York, 1960, 340 pp., $9.50

This modern treatment of the principles of structural analysis is in full accord with the mathematical trend in the study of the engineering sciences. The authors stress that structural design, as an art, must be built on a firm scientific foundation. Their book develops a sound basis for the solution of structural problems encountered in practice.

The basic theory is applied not only to traditional structures such as roof trusses and bridges, but also to aircraft structures and machine design.

General methods are emphasized, rather than techniques of limited application. The method of influence lines is discussed thoroughly, and graphic statics is extended to problems much more complicated than those met in courses in mechanics.

Extensive coverage is given of three-hinged arches and of the effects of moving loads on frames. The treatment of space truss structures, evolving from German and French research, is extensive.

Ultimate strength calculation of reinforced concrete (Le calcul du béton armé a la rupture)

R. Chambaud, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 158, Feb. 1961, pp. 185-237

Presents experimental data verifying the method of ultimate strength design presented by the author in *Annales*, Mar.-Apr. 1959. Correlation was established between observed bending moments and calculated moments for a large number of beam specimens broken in bending.

Configuration of specimens, dimensions, characteristics of the materials, and percentages of reinforcement were varied over a wide range to validate the conclusion that the proposed method is applicable to usual cases of simple bending.

Plate restrained at four edges, supported by elastic beams, subject to uniformly distributed load (Die Vierseitig starr eingespannte Platte auf elastischen Trögern unter Gleichlast)

H. Bechert, *Beton und Stahlbetonbau* (Berlin), V. 56, No. 1, Jan. 1961, pp. 14-20
Reviewed by Rudolph Szilard

An approximate method for the determination of the influence of the elastic supports has been derived using Homberg's mathematical model of a gridwork. A numerical example yields the following results: the conventional assumption of rigid support is safe in most of the cases; to arrive to an economical design the continuity between individual plates should be considered.

The British Standard Code of Practice for the use of normal reinforced concrete in buildings (CP114-1957)

H. G. Cousins, *Reinforced Concrete Review* (London), V. 4, No. 11, Sept., 1958, pp. 679-703
Reviewed by William R. Lorman

Presents highlights of the revised code (CP114-57) which was published in 1957 under the authority of the British Standards Institution. States the reasons for revision and comments on the effects of these revisions as related to: computational symbols, concrete mix proportioning, permissible shear and bond stresses, reinforcement (cover, bond, anchorage), buckling, bending moments, slabs, stairs, columns, walls, and methods of testing. Compares load-factor method and elastic method of design with respect to rectangular beams, T-beams, and columns. The extended discussion, by 20 experts, also includes comparisons concerning concrete design practices in the United States and England. Reviewer is not in a position to attempt any correlation of the discussion content with the content of the "ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-56)" for reasons that are reflected in a remark made by one of the discoursers, to wit, " . . . what a gruelling time it had been for the author trying to answer in a matter of seconds questions which the speakers had probably been working out all the week!" In all seriousness, the reader who is interested in this paper by Cousins in all likelihood is interested in Baker's paper entitled "The Work of the European Committee for Concrete" which is reviewed separately.

Practical design of structural members

Thomas A. Lucy, F. W. Dodge Corp., New York, 2nd Edition, 1961, 432 pp. $12

Emphasizes practical, workable results intended to enable the designer to think in terms of structures rather than mathematical equations.

The book contains a large number of tables, charts, and diagrams supplemented by comprehensive analyses and discussions. Among the members and connections examined are: reinforced concrete—beams and slabs, columns, foundations and footings, retaining walls, and masonry; structural steel—beams, plate girders, columns, rivets and connections, angles, roof trusses, joists, and steel deck; wood—beams and joists, flooring and sheathing, stud partitions, and posts; plus thorough analyses of rigid frames, wind stresses, and earthquake resistance.

Analysis of frames with V-supports

K. Starzewski, *Concrete and Constructional Engineering* (London), V. 56, No. 4, Apr. 1961, pp. 141-145

Author's Summary

Frames with V-supports are used in bridge construction and less frequently for long-span roofs. Unless the slope of the top member of a roof frame is steeper than 1:10 it can be neglected. When the inclined columns are slender their flexural stiffness can be neglected. In general, however, the flexural stiffness of the columns should be taken into consideration. An example is given in the article of the analysis of a symmetrical frame by the moment-distribution method of analysis; the usual convention of signs is used.

The method of determination of deflections of reinforced concrete slabs supported at the circumference (in Russian)

A. N. Kobolev, *Beton i Zhelezobeton* (Moscow), No. 3, Mar. 1960, pp. 138-141
Applied Mechanics Reviews V. 14, No. 5, May 1961

Method applies to slabs after appearance of cracks. As is well known, deflections of slabs in this state cannot be found by the theory of elasticity of homogeneous plates.

Author's method is based on Murashov's theory (1940) of stiffness in bending of reinforced concrete beams after cracks have appeared, and on the principles of ultimate load design, applied to plates.

The values found by the applied method agree fairly well with experimental results obtained by Bach and Graf (1915).

Statically indeterminate structural analysis

R. L. Sanks, Ronald Press Co., New York, 1961, 602 pp., $10

A thorough treatment of indeterminate analysis by moment area, virtual work, and moment distribution. Other methods are presented in less detail but with adequate explanation for evaluation of their value.

By a meticulously developed extension of the moment-area method to frames, the usefulness of this method has been greatly expanded. The concept of the cut-back structure is used to shorten the work of calculation of deflections in statically indeterminate structures. An unusually thorough treatment of the subject of elastic checks has been included.

In the final section, the author discusses the use of models in structural design and analysis. Many difficult structural examples are presented, together with careful step-by-step solutions. The book has many line drawings, with problems for solution, and with the answers to selected problems.

Elementary statics of shells

Alf Pfluger, F. W. Dodge Corp., New York, 1961, 122 pp., $8.75

A visual presentation of the spatial interplay of forces in shells which differ considerably from the static behavior of linear members. The author presents the general concepts of shell theory and defines the basic terms. He then gives the basic assumptions which justify his visual approach to design. These assumptions lead to the central thesis of the book, viz., the membrane theory.

The membrane theory is then applied to every general type of shell: spherical, toroid, conical, circular cylindrical, paraboloid, barrel vaults, and several combinations. Theory is elaborated only when necessary for the solution of the more common problems. All equations are capable of visual interpretation and their results can be interpreted in tables and graphs. A tabulation of membrane theory solutions adds to the usefulness of the book.

Tables for ultimate strength design of reinforced concrete

James M. Cunningham, Haley W. Keister, Arthur W. Newett, Jr., and James R. Phillips, CKNP Tables, P.O. Box 16434, Jacksonville 16, Fla., 1961, 177 pp., $4.95.

Contains tables computed in accordance with the ACI 1956 Building Code and based on 3000 psi concrete and 40,000 psi deformed bars. Instructions are included for adjusting tabulated values for use of lightweight aggregate concrete. A wide range of sizes of members and reinforcement are included to make the tables useful for estimating, preliminary design, and final design.

Basic theories and derivations of ultimate strength design formulas are given for those who wish to solve problems by computation. Ultimate strength wind formulas graph and derivation are intended for determining which load combination formula governs when wind is a design factor. Numerous other examples add to the usefulness of the book.

Wind structure and the assessment of wind loads

WALTER A. WOEBER, *The Structural Engineer* (London), V. 39, No. 5, May 1961, pp. 155-162

Relates the problems associated with the assessment of wind loads in the design of exposed engineering structures with that part of dynamic meteorology which deals with atmospheric motion near the ground. Appraisal of the complex structure of natural gusty wind indicates that load classifications in many building codes do not accurately represent actual conditions. Author's method extends routine evaluation of anemograms at weather stations and is offered to discourage use of arbitrary rules.

Composite construction in theory and practice

KONRAD SATTLER, *The Structural Engineer* (London), V. 39, No. 4, Apr. 1961, pp. 124-144

Thorough treatment of the subject with the primary purpose of describing composite construction embracing steel beams, both with and without prestressing. Mention is made as to when the bases of calculation can also be applied to prestressed concrete construction. The section on theoretical consideration includes calculations required for creep and shrinkage and also discusses idealized cross sections; statically determinate and statically indeterminate composite sections with solid webs are covered, as well as composite construction with lattice girders. Although not often critical, the author discusses clarification of load factor. After touching briefly on shear forces and connectors he discusses elastic connections and high strength friction grip bolts.

The second section on practical applications is extensive, discussing first shear connectors and then going into various applications in structures, including various types of bridges. Examples of many different actual projects are included to demonstrate the wide range of application of composite construction.

Addendum to the elastic supports of structures (Beitrag zur elastischen Stützung von Bauwerken)

B. GOSCHY and GY. BALAZS, *Beton und Stahlbetonbau* (Berlin), V. 56, No. 1, Jan. 1961, pp. 10-13

Reviewed by RUDOLPH SEILARD

Spring contents and other elastic properties of the soil are determined. Effect of the elastic foundation has been derived using the classical method for solving statically indeterminate structures. The reactions are a function of the time because of the long time foundation settlements.

Rapid design of continuous beams by Caquot's method (Calcul rapide des poutres continues par la méthode de M. Caquot)

MARCEL REIMBERT and ANDRE REIMBERT, Editions Eyrolles, Paris, 1960, 260 pp., 34 NF

Reviewed by ARON L. MISSKY

Albert Caquot's method, which has been adopted by the French ministry in charge of reconstruction and urban planning as the best, simplest, and yet most rigorous of the many available "simplifications" of continuous-beam analysis and design, starts with the classical theorem of three moments; a structure continuous over n spans is converted into a series of $(n-1)$ two-span girders; for each, substitution of $l'_L = 0.8\ l_L$ and $l'_R = 0.8 l_R$ renders the end moments M_L and M_R equal to zero, so that a simple relation becomes available for the intermediate moment M_M (the numerical coefficient being arbitrarily adjusted to give better accuracy).

Book is designed to simplify the design engineer's task. First part (pp. 19-142) evaluates many cases of distributed and concentrated loadings, including moving loads and influence lines (pp. 83-142), from the theoretical standpoint, while second part (pp. 143-218) gives many practical examples, including the design of the reinforcement and the preparation of preliminary designs. Third part (pp. 219-260) is a well-stocked formulary.

Materials

Effect of partial replacement of cement with surkhi on various properties of pavement concrete

M. L. Puri and N. R. Srinivasan, *Road Research Papers*, Central Road Research Institute of India, New Delhi, No. 44, 1959, 24 pp.

Considerable work has been carried out in investigating partial replacement of cement with pozzolanic materials in concrete for hydraulic structures; little data is however available on pavement concrete utilizing pozzolans. This study was conducted to obtain data on some of the properties of concrete (workability, flexural strength, compressive strength abrasive resistance, resistance to alkali-aggregate reaction, and sulfate attack) for concrete to be used in pavements. Four main types of clays (surkhis) were studied.

Investigation of nonmetallic waterstops: Evaluation of the effect-of-alkalies and accelerated extraction tests

Leonard Pepper, *Technical Report* No. 6-546, Report 2, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Mar., 1961, 28 pp.

Author's Summary

Ninteen nonmetallic waterstop materials (polyvinylchlorides, polyethylenes, and natural or synthetic rubbers) were subjected to the accelerated extraction and effect-of-alkalies test required by Corps of Engineers specifications for polyvinylchloride waterstops. The former test affected (a) the tensile strength of all the materials except standard grade polyvinlychloride, and (b) the ultimate elongation of all the materials. In the effect-of-alkalies test, changes in weight and hardness were linear for the two times of observation; changes in thickness were due to experimental error only; all materials met the change-in-hardness requirement. Most of the rubbers failed to meet the accelerated extraction test requirements and the effect-of-alkalies test change-in-weight requirement. Based on these results, it is recommened that the Corps of Engineers specification be changed by: (a) deleting the change-in-thickness portion of the effect-of-alkalies test; (b) limiting the change-in-weight test to one observation with three replications; and (c) clarifying the specification limits for both tensile strength and ultimate elongation.

Glass reinforced plastics for reinforcing concrete structures (in Russian)

A. A. Gvozdev, K. V. Mikhailov, and I. Nikula, *Beton i Zhelezobeton* (Moscow), No. 3, Mar. 1960, pp. 105-111

Applied Mechanics Reviews V. 14, No. 4, Apr. 1961

The economy of steel for concrete reinforcing requires new materials, because in 1965 (in USSR), according to the planned estimate, 6,000,000 tons of steel reinforcing will be required. In a particular case, dielectric concrete structures are required, which in turn make it necessary to avoid the use of steel for reinforcing. Materials of the nylon type cannot be used, because, despite their high strength up to 8000 kg per sq cm, their moduli of elasticity are as low as 35,000 to 50,000 kg per sq cm, the temperature expansion is up to 10 times that of concrete, and they are subject to large creep. Only the invention of a new material for concrete reinforcing may solve the problem. The use of glass appears possible for this purpose. It has strength up to 22,000 kg per sq cm with unit weight 3.5 times smaller than that of steel, with an elastic modulus up to 800,000 kg per sq cm. It appears that difficulties for its use are avoidable.

As a rule, the reinforcing glass must be used in sandwiches inside of plastics similar to post-tensioning cables. Such material is manufactured by Leningrad "Factory of Sandwich Plastics." The writers give technical characteristics and mechanical properties of these plastics. They describe their research on experimental plastic-concrete beams. Due to a low modulus of elasticity of the glass-plastic sandwich ma-

terial, most beams were prestressed. The relaxation losses were investigated. Results of testing of these beams are represented by comprehensive graphs and tabulated data.

Authors conclude that the type of structures investigated can be applied for practical purposes; economically, however, they cannot compete with ordinary reinforced concrete. More researches are necessary. The four research institutions in USSR already busy with similar researches must increase their effort to obtain better plastic-reinforced concrete structures. Generally, the structural elements must be prestressed.

ASTM standards on cement (with related information)

American Society for Testing Materials, Philadelphia, 15th Edition, 1960, 286 pp., $4

A handy collection of ASTM specifications, chemical analyses, and physical tests related to cement sponsored by ASTM Committee C-1. To these reprints from the ASTM Book of Standards have been added information on analytical balances and weights, a manual of cement testing a list of selected references on portland cement, personnel of Committee C-1 on Cement and its subcommittees, and the committee bylaws.

The production and quality control of sand and gravel aggregates

B. L. Morton, *Reinforced Concrete Review* (London), V. 4, No. 9, Mar. 1958, pp. 505-522
Reviewed by William R. Lorman

Describes current commercial and planning considerations for the working of gravel deposits; also treats the future problem of aggregate reserves. Excavation and processing methods are presented. Fine and coarse aggregate classifications and quality controls are dealt with briefly. Basic theme points to the necessities for improving standards in the aggregate industry. The views of nine contributors comprise the discussion.

Methods for sampling and determining sieve analysis of aggregates larger than 1½ in. sieve.

R. L. Curry, R. E. Phillco, and Bryant Mather, *Technical Report* No. 6-543, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., May 1960, 25 pp., $0.50
Author's Summary

Aggregates in the 1½ to 3 in., and the 3 to 6 in. size ranges were sampled by the following methods: clamshell from stockpile, trenching loaded trucks, from conveyor belt loading bins, and during bin discharge. All samples were tested for sieve analysis. It is concluded that: any of the sampling methods are satisfactory; 100-lb samples of 1½ to 3-in. aggregate and 300-lb samples of 3- to 6-in. aggregate are large enough; diameter of wire in 5- and 6-in. sieves should be 0.360 to 0.530 in.; hand-sieving aggregate coarser than 3-in. sieve can be satisfactorily accomplished. Three appendices are included: a revised test method for sieve analysis of aggregate; revisions to the method of sampling aggregate; and a discussion of sample sizes.

Progress in ceramic science — V. 1

J. E. Burke, Pergamon Press, Inc., New York, 1961, 232 pp., $10

A review of the state of knowledge in chemistry and physics related to ceramic materials and processes. These annual volumes are intended to provide authoritative reviews of the various aspects of the field to aid engineers and scientists in defining and solving critical problems. Of particular interest to workers in the cement field is the chapter on chemistry of cement hydration by H. F. W. Taylor, Department of Chemistry University of Aberdeen, Scotland.

Emphasis is on the nature, proportions, structures, and properties of the various phases formed on hydration of cement, rather than on the mechanisms or thermochemistry of the hydration processes. The first part of the review deals with the fundamental chemistry and crystallography of hydrated calcium silicates and aluminates; the se-

cond with the hydration of portland and aluminous cements at normal temperatures; and the third with steam-curing of portland cement, both alone and in the presence of reactive silica.

Lightweight aggregate concrete made with foamed blast furnace slag (Leichtbeton aus Hüttenbims)

L. KNUBBEN, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 10, Dec. 1960, pp. 597-600

Reviewed by FERDINAND S. ROSTASY

If molten blast furnace slag is foamed and chilled with water or compressed air, a lightweight aggregate is obtained. Author reports of tests conducted at the Research Institute of the German Cement Industry regarding the properties of concrete made with this aggregate. The dry weight of the concrete was about 100 lb per cu ft. Compressive strength determined on 6 x 12-in. cylinders was 4000 psi maximum at 28 days. Cement contents varied between 4 and 8 sacks per cu yd. The concrete showed high resistance to freezing and thawing. Final shrinkage was less than 1 percent.

Influence of elevated cement and concrete temperatures (Uber den Einfluss erhöhter Zement- und Betontemperaturen)

J. BONZEL, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 11, No. 3, Mar. 1961, pp. 192-194

Reviewed by FERDINAND S. ROSTASY

Author discusses the question of the upper limit of temperature of fresh concrete. In times of strong construction activity often hot cement is used which involves the danger of false set. It is shown that knowing the original temperatures of the components of the fresh concrete, the temperature of the latter can be calculated. The author stipulates that the fresh concrete temperature should be below 86 F. The cement temperature is of less influence than the temperature of the aggregates. It should, however, remain below 18 F. Cooling and drying out of the fresh concrete should be retarded by suitable measures.

Hypocalcic cements (Sur les ciments hypocalciques)

F. FERRARI, *Revue des Matériaux* (Paris), No. 544, Jan. 1961, pp. 20-22

Reviewed by PHILLIP L. MELVILLE

A summary on the investigation of pozzolanic cements and on the need for manufacture of a low calcium fondu cement with pozzolans.

Self stressing of concrete by hydration of magnesia (L'autocontrainte des bétons par l'hydratation du periclase MgO)

V. SLAFANOFF and N. DJABAROFF, *Revue des Matériaux* (Paris), No. 544, Jan. 1961, pp. 30-33

Reviewed by PHILLIP L. MELVILLE

Self stressed concrete using expensive cements is discussed with application to pipe manufacture. Magnesia was added to portland cement and on test specimens the expansive force developed was from 9000 to 12,000 kg per sq cm in the steel. Steam curing will expedite expansion. No corrosion problem was found.

Protecting concrete against corrosion

H. R. TOUCHIN, *Corrosion Technology*, V. 4, No. 12, 1957, pp. 417-419

Reviewed by WILLIAM R. LORMAN

Presents information concerning the most aggressive types of compounds that attack concrete. Points out the merits and failings of bituminous solutions, oils, varnishes, and chlorinated rubber paints as protective coatings for concrete. Emphasizes the properties and effectiveness of epoxide resin coatings, the potential reduction of mold growth, and the relative ease of maintenance attainable with concrete surfaces so treated. Nevertheless, your reviewer cannot help but wonder why the largest grain elevators (concrete) on the Pacific coast (Stockton, Calif.) were given a ¼ in. thick protective coating of "standard aluminum asbestos," as described on p. 416 opposite the article reviewed, when epoxide resins supposedly possess so many attributes; apparently reduction of solar

heat transmission is not one of the virtues of epoxide resin coatings applied to concrete surfaces. The reader is referred to a previous review* for additional comments concerning epoxide resins for concrete protection.

*"Current Reviews," ACI JOURNAL, *Proceedings*, V. 54, No. 7, Jan. 1958, p. 623.

Prestressed Concrete

Prestressed roof trusses in the German Democratic Republic (Vorgespannte Fachwerkbinder in der Deutschen Demokratischen Republik)

SIEGFRIED MANLEITNER, *Bauplanung-Bautechnik*, V. 14, No. 11, Nov. 1960, pp. 493-498

Field assembled prestressed concrete roof trusses with curved top chord in the German Democratic Republic (Zusammensetzbare Spannbeton-Fachwerkbinder mit gekrümmtem Obergurt in der Deutschen Demokratischen Republik)

SIEGFRIED MANLEITNER, *Bauplanung-Bautechnik*, V. 15, No. 1, Jan. 1961, pp. 8-12
Reviewed by J. F. LEPPMANN

Construction of roof trusses of prefabricated elements has only recently been introduced into East Germany. Three systems of various degrees of subdivision have been investigated: (a) all truss members are individually prefabricated, (b) truss members are precast in groups forming single or multiple triangles, or (c) combinations of (a) and (b). Prestressing of the bottom chord is intended in all cases. Costs of fabrication, storage, and transportation decrease but erection difficulties and costs increase with greater fragmentation.

The first paper discusses in detail the design of a 100-ft truss for both Types (a) and (b) and describes the use of the Type (b) truss on a pilot project.

The second paper deals with problems of standardization. Economical competitiveness of this type of construction depends on mass fabrication of the component parts. Its chances are improved since the interested organizations of the Iron Curtain countries agreed to restrict roof spans of industrial buildings to 60, 80, 100, or 120 ft. Proposed for standardization is a system combining elements of (a) and (b), using 20-ft panels and L/f ratios of about 8 with two groups of dimensions, satisfying the requirements of either the 80-ft or the 120-ft span, and with such details that 60-ft and 100-ft trusses can be formed by omitting the center panel of the next to larger size, lowering the necessary 28-day concrete cube strength from 8400 to 6300 psi and reducing the number of tendons in the bottom chord.

Prestressed concrete railway viaduct in Silesia (in Polish)

W. PIETURA, *Inzynieria i Budownictwo* (Warsaw), No. 1, 1960, pp. 25-34
POLISH TECHNICAL ABSTRACTS No. 3/4 (39/40), 1960

The three-span viaduct, 72 m long, is the first prestressed concrete structure built in Poland. The main constructional element consists of three simply supported beams, each 2 m high. Length of spans is 17.5, 28.1, and 26.8 m. A description is given of the individual elements as well as basic data concerning the strength and loading, and a description of the assembly. Tests included trial loading of the viaduct.

The allowable F_o-e region for prestressed sections and an exact method for optimum prestress force modification

K. K. KIENOW and C. A. BRYAN, *Journal*, Prestressed Concrete Institute, V. 6, No. 1, Mar. 1961, pp. 22-28
AUTHORS' SUMMARY

Equations for the curves $\alpha\beta\gamma\Delta$ which define the region of allowable sets of (F_o-e) values for a prestressed composite section are developed in terms of the section properties, load moments, and allowable stresses. A direct method of prestress modification is developed for cases in which the optimum F_o-e cannot be physically obtained.

Prestressed concrete — Design and construction

JAMES R. LIBBY, The Ronald Press Co., New York, 1961, 468 pp., $12.50

Following introduction to the basics of prestressed concrete, the reader is lead into the development of flexural analysis and practical methods of design. The explanations accompanying the various design problems are particularly helpful.

More than ordinary attention is given to prestressing systems and procedures and to facilities of plant and fabricating procedures. Available plant and equipment necessarily affects selection of system and type of construction.

The manufacture of hollow reinforced concrete beams (Dow-Mac process)

W. K. ROBERTS, *The Reinforced Concrete Review* (London), V. 4, No. 5, Mar. 1957, pp. 329-347

Reviewed by WILLIAM R. LORMAN

Describes various processes employed in the manufacture of long-span prestressed hollow beams. Production of heavier cross section units suitable for use as hollow bridge beams or hollow piles is described also. Prestressing is accomplished in accordance with the Hoyer system. Details relative to molds, casting, tensioning and detensioning of wires, demolding, and erection are included; these various features are depicted by 16 excellent photographic views. The discussion, which covers questions or comments of 14 individuals, pertains mainly to the technical details of the various phases comprising production of these hollow units. The reader who is interested in the production of the American precast hollow unit known as Flexicore will find this paper (and its discussion) elucidating. Reviewer recommends that such reader refer to *Civil Engineering*, V. 23, No. 11, Nov. 1953, pp. 760-764 for reciprocal information that is concerned with the American units used at the U.S. Naval Station, Great Lakes, Ill.

Prestressed concrete — A review

F. WALLEY, *Reinforced Concrete Review* (London), V. 4, No. 10, June 1958, pp. 609-635

Reviewed by WILLIAM R. LORMAN

Reviews developments that have occurred, both in Britain and elsewhere, since 1946 relative to research and practice in the area of prestressed concrete. The principal divisions of this paper are: (a) design methods and assumptions, including prestress loss due to wire relaxation and fire; (b) steel and concrete; (c) stressing techniques, including pretensioning, posttensioning, circular stressing, and flatjacking; and (d) structures, including general buildings, bridges, and miscellaneous items (e.g., prestressed tanks and piles). Includes three structural design illustrations, eight photographs of applications, and eight references. Though concrete creep and shrinkage are merely mentioned in the paper, such features receive somewhat more consideration in the discussion which reflects the views of 11 individuals.

Reinforced concrete joints between prestressed concrete members

J. S. REEVES and P. B. MORICE, *Magazine of Concrete Research* (London), V. 13, No. 37, Mar. 1961, pp. 13-20

AUTHOR'S SUMMARY

The practice has developed in recent years of constructing statistically indeterminate prestressed concrete structures by applying connecting restraints to two or more statically determinate prestressed concrete members. This paper discusses a joint that may be used to convert two or more simply supported prestressed concrete beams into a continuous beam by joining them together over the supports. This joint uses unprestressed continuity steel contained within the cross section of the continuous beam.

Alternative designs for such a joint are proposed and examined for suitability in fulfilling the necessary functions. Some remarks are added on the design of composite continuous beams.

Tests are reported on five two-span composite beams each made from two

post-tensioned, grouted beams. The ratio of the ultimate moments of resistance of the critical sections was varied. The tests demonstrated the essentially satisfactory nature of the proposed joint.

Properties of Concrete

Shielding properties of ordinary concrete as a function of temperature

E. G. Peterson, *Report* HW-65572, United States Atomic Energy Commission, Aug. 2, 1960, 42 pp., $1 (Available through Office of Technical Services, Department of Commerce, Washington 25, D. C.)

Nuclear Science Abstracts
V. 15, No. 9, May 15, 1961

The effect of high temperature on neutron attenuation in concrete was determined. The fast neutron attenuation was exponential and could be adequately predicted using removal theory. The calculated removal cross sections agreed with the measured values to within 3 percent. Tests indicated that even after penetrating 47.5 in. of concrete the thermal and resonance neutron fluxes had not reached equilibrium with the fast neutron flux. Significant changes in neutron flux values were observed after the concrete was baked at 100, 200, and 300 C. A combination of removal and age theories adequately predicted the increase in the fast, resonance, and thermal neutron fluxes. The measured relaxation lengths and removal cross sections are summarized. Changes in reflector and thermal shield thickness or composition from test values would affect the relaxation length. The shield facility was calibrated so that removal cross sections can be measured. The dose rate generated through the full thickness of the as-cured specimens was primarily due to the fast neutrons. After the concrete was baked at 300 C, the dose rate was a result of resonance neutron leakage. The γ dose rate was measured through 47.5 in. in the 100 C test only. The data indicated that the biological dose rate through 47.5 in. of concrete is primarily due to γ leakage at this temperature.

Concrete shrinkage (in Swedish)

W. Czernin, *Cement* (Malmö), V. 35, No. 2, 1960, pp. 115-130

Road Abstracts
V. 28, No. 1, Jan. 1961

Paper defines the term, distinguishing between physical and chemical shrinkage occurring in fresh concrete, and then discusses the mechanism of shrinkage, the most important relevant factors (cement quality, water-cement ratio, aggregate, drying out, temperature, carbon dioxide) and crack formation. A list of 21 references is appended.

Studies in reinforcement-concrete bond

J. A. Roberts and H. E. Vivian, *Australian Journal of Applied Science* (Melbourne), V. 12, No. 1, Mar. 1961, pp. 104-130; 131-139

Author's Summary

I. Improvements in bond

A number of processes by which steel-concrete bond can be improved are described. The procedures involve coating steel with an air or thermal setting and hardening resin such as an epoxy resin and either casting the concrete around the treated reinforcement before the resin hardens or applying sand to the unhardened resin which is then allowed to harden before the concrete is cast. Bond determined by pull-out tests is increased by up to 500 percent at various ages up to 84 days after fabrication of specimens.

II. Behavior of reinforcement-concrete bond in the presence of calcium chloride additive and under steam-curing conditions

The effect of calcium chloride additions to concrete as well as the effect of steam curing on the improvement of bond between concrete and resin-coated steel have been studied by pull-out tests. Increases in bond of up to 500 percent, compared with that of untreated steel, have been obtained at various ages up to 84 days. Severe corrosion of steel was observed only on untreated steel bars embedded in steam-cured concrete that contained added calcium chloride; a slight surface modification was observed on resin-tested steel under the same test conditions.

Thermal stress in concrete

I. W. Hannah, *Nuclear Engineering* (London), V. 6, No. 57, Feb. 1961, pp. 69-74
Ceramic Abstracts
V. 44, No. 5, May 1961

During construction of a biological shield, laboratory scale research was carried out to determine the nature and effect of thermal stresses in concrete. Preliminary results indicate that the ultimate load characteristics in bending, of sections reinforced with more than 0.3 percent steel, are largely unaffected by thermal stress caused by moderate linear temperature gradients.

Conventional building materials as protective radiation barriers

E. Dale Trout, John P. Kelly, and Arthur C. Lucas, *Radiology* V. 76, Feb. 1961, pp. 237-244
Nuclear Science Abstracts
V. 15, No. 9, May 15, 1961

A study was made of the radiation shielding provided by various conventional building materials. An x-ray unit, provided with a diaphragm that could be adjusted for square or rectangular fields, was used as the source. Structural panels of eight typical wall materials were tested. These panels ranged in thickness from a thin gypsum plaster wall to a thick cinder block and portland cement mortar wall. Radiographs were made of representative areas of each wall section. The transmission data were converted to lead equivalents and also to concrete equivalents. A schematic sketch is included of a typical radiographic room presenting typical shielding situations. Data are tabulated on the thickness of lead which would have to be added to a particular wall to provide adequate shielding when using the various types of construction. It is concluded that the shielding effect of masonry walls is sufficient to provide useful barriers for many applications. Walls of conventional building materials should be considered in shielding calculations even though they may not provide all the shielding necessary in a given wall. A considerable reduction in the amount of lead required will usually be effected.

Behavior of heat resistant reinforced concrete units subjected to compression or tension and nonuniform heating (in Russian)

A. F. Milovanov and V. S. Zibyanov, *Beton i Zhelezobeton* (Moscow), No. 7, July 1960, pp. 310-316
Applied Mechanics Reviews
V. 14, No. 4, Apr. 1961

One surface of 25 x 25 x 250 cm reinforced concrete beams (made using a heat resistant aggregate) has been heated to 300, 500 and 700 C. At those temperatures beams were subjected to axial forces until failure. Crack formation and deformation of beams were observed. It was found that Young's modulus of a heated concrete decreases. No influence of the applied heating regime on the carrying capacity of tested units has been observed.

Development of an apparatus to determine the resistance to segregation of fresh concrete

Barry P. Hughes, *Civil Engineering and Public Works Review* (London), V. 56, No. 658, May 1961, pp. 633-634

Brief description of an apparatus to measure the resistance of fresh concrete to segregation and this provides a means of determining its cohesiveness. Use of the apparatus is seen for evaluating the effects of gap-grading and for developing stability factor values for different placing conditions to be used in a laboratory test of mix proportions.

Effect of the size and shape of a concrete test body on its strength (in Polish)

S. Bastian, *Inzynieria i Budownictwo* (Warsaw), No. 1, 1960, pp. 13-18
Polish Technical Abstracts
No. 3/4 (39/40), 1960

Tests were made with concrete cubes of side measurements of 7.1, 5.68, 4.26, 2.84, and 1.42 cm. The studies covered two basic parameters: technological parameters (effect of the type and quantity of cement in the mortar, effect of time elapsed between casting and testing) and geometrical parameters (effect of the size and shape of the body). The effect of the slenderness of a body on its strength increased with a

rise in the ratio between the total surface and the volume of the test body. A formula is given for computing the strength of a cube with a side of 7.1 cm on the basis of the strength of a test body of smaller dimensions.

Structural Research

Problems and possibilities of concrete shells

E. Torroja, *Reinforced Concrete Review* (London), V. 4, No. 12, 1958, pp. 729-738
Reviewed by William R. Lorman

With regard to thin shell concrete structures, the author points out the advantages of experimental stress analysis in comparison with pure mathematical analysis. Explains the unique construction features of concrete shells built in conjunction with suspended steel mesh.

Some experiments with concrete

E. H. Bate, *Reinforced Concrete Review* (London), V. 4, No. 7, Sept. 1967, pp. 421-447
Reviewed by William R. Lorman

Describes investigations sponsored by the British Ministry of Works. Reports conclusions of recent tests conducted with the view of improving the quality of concrete used in the Ministry's construction programs; a large portion of such construction is accomplished by contracts with medium or small firms that cannot afford to perform research of their own accord. Tests were carried out in the following areas: (1) strength of construction joints, (2) design of reinforced concrete beams, and (3) design of concrete mixtures. Reinforcement layout detail comprises the first two figures and an improved horizontal construction joint schematic comprises the third figure; the four tables of test data reflect the three experimental areas investigated. In the discussion, which also contains the remarks of 14 individuals, the author amplifies the paper with details pertaining to construction joint practice, concrete mix proportioning variations, and water leakage in basement construction.

Earthquake effects on structures in Chile (Temblores Chilenos de mayo de 1960; efectos en estructures civiles)

E. Rosenblueth, *Ingenieria* (Mexico City), V. 31, No. 1, Jan. 1961
Reviewed by Manuel Renasco

A complete study has been made of earthquake effects on all types of structures in Chile. Failures have been analyzed in foundations, soils and settlement. Effects on all types of structures from concrete roadways to seven story buildings. Tables are developed with seismic coefficients but not including effects of torsion. Failures were classified due to: flexure, diagonal tension, and column torsion. Complete list of all failures and apparent causes.

The use of lightweight concrete for reinforced concrete construction

A. Short, *Reinforced Concrete Review* (London), V. 5, No. 3, Sept. 1959, pp. 141-187
Reviewed by William R. Lorman

Presents latest developments pertaining to design and research in structural lightweight concrete made with lightweight aggregates; cellular or aerated concrete is not considered at length. In describing the suitability of aggregates, the following are dealt with: foamed blast-furnace slag, sintered clay or shale, sintered pulverized fuel ash, expanded slate, and pumice. The survey of codes of practice covers ASTM C-330, ACI Building Code (ACI 318-56) and the British Standard Code of Practice (CP 114-57). Sections pertaining to corrosion of embedded steel reinforcement, bond strength, and beam performance precede the section on permissible design values. The paper contains two tables of aggregate properties, 16 figures of which 12 are graphical, and 33 references. The discussion, which reflects the opinions of 14 authorities, is concerned mainly with corrosion and bond of reinforcement embedded in lightweight concrete. This paper, and its discussion, offers the reader a good picture of current trends in this area of concrete. Abeles's comments (p. 186) regarding creep and

shrinkage would appear to indicate that such volume changes may be accounted for satisfactorily when designing lightweight concrete structures.

Investigation of cylindrical columns with prestressed spiral reinforcement (in Russian)

F. E. Gitman, *Trudi Nauk-i. In-ta Betona i Zhelezobetona* (Moscow), Akad. Str-va i Arkhitekt. SSSR, No. 3, 1958, pp. 204-236; *Referativnyi Zhurnal Mekhanika*, No. 9, 1959, Rev. 10898

Applied Mechanics Reviews
V. 14, No. 5, May 1961

Ferroconcrete columns were examined: (1) uniform and thick-walled, with different percentages of longitudinal reinforcement in a spiral winding, with different intensities of prestressing in two variants of finish for the surface—with a protective layer and without; (2) thin-walled with a constant percentage of longitudinal reinforcement in a spiral winding with different intensities of prestressing. With a constant outer diameter of the columns of 35 cm, the thickness of the walls for the thin-walled columns was 3.5-4.0 cm and for the thick-walled 12-13 cm. The uniform columns were made by filling the thin-walled columns with vibrated concrete. The test pressures were developed in presses with powers of 500 to 1000X. The columns on the press were centered using strain gages located in quarters of the length at loads of about one quarter of the loads required for disruption. The measured axial and radial deformations were recorded in the form of graphs while the data for carrying capacity were incorporated in a table.

The author worked out formulas for the calculation of the columns for central compression on the assumption that in the transverse sections of the uniform and thick-walled columns the attainment of plasticity precedes disruption over the whole section, while in the thin-walled columns the diagram representing the stresses in the walls is triangular. Some deductions are furnished which are of interest in theoretical and practical aspects.

Strength tests of walls of gas-concrete block (in Polish)

J. Zembrowski, *Archiwum Inzynierii Ladowej* (Warsaw), No. 2, 1959, pp. 169-213

Polish Technical Abstracts
No. 3/4 (39/40), 1960

Presents the results of tests on the load carrying capacity and buckling of walls built from 24 x 24 x 49 cm cellular concrete block. Several types of cellular concrete were used; the strength range was 16 to 64 kg per sq cm. Test results are included for 27 sets of columns, using various mortars to assemble the units, and about 1000 sets of compression test pieces. After showing that the formula for strength of concrete masonry cannot be applied to cellular concrete units, the author establishes an equation for the load carrying capacity of such a wall, characterized by the introduction of, in addition to the compressive strength, the tensile strength. A method for statistical treatment of the results is discussed. Using the equation established for the elasticity of a wall and the test results of slender columns, the author verifies the equation for the buckling factor. The author also establishes an equation for the safety factor of a wall.

Comparison test of reinforcing steels

R. H. Carlson and J. P. Mumma, *Report* WT-1473, Civil Effects Test Group, U. S. Atomic Energy Commission, 1960, 108 pp., $2.25 (available from Office of Technical Services, Department of Commerce, Washington 25, D. C.)

Author's Summary

Describes an experiment conducted during the Operation Plumbbob nuclear tests in 1957.

Project 34.2 was planned to determine the relative merits of rail and intermediate grade steel as reinforcement in concrete slabs subjected to blast loading. Slab pairs were placed at ground level over deflection chambers and loaded with the incident pressure pulse from a nuclear device. Two station locations were chosen so that the loadings were approximately 7 and 5 psi peak overpressure. The slabs comprising each pair were identical except for the grade of reinforcement in each.

Slab strengths were varied, and two test stations were used to increase the probability of realizing the response necessary to emphasize the advantages of each steel grade.

Measurements were made of the dynamic midspan slab deflections and the transient loading pulse. A thorough post-test survey was conducted to determine the severity of the resulting damage to the concrete slabs.

Test results indicated that the desired range of slab behavior was achieved. In all cases the rail slabs displayed the more desirable response. However, limitations of this experiment, due to loading and structural conditions, must be considered before this conclusion can be generalized.

Methods adopted in the USSR for the study of fire resistance of structural elements and some results obtained to date

V. I. Murashev, unidentified Russian monograph, 1959, 20 pp. (English translation No. 60-10734 available from Library of Congress or SLA Translation Center: microfilm $2.40, photocopy $3.30)

Technical Translations
V. 3, No. 5, Mar. 9, 1960

Limit fire resistance of simply supported reinforced concrete elements is attained at the moment when a plastic hinge is formed (when the elastic limit of the tensioned steel is reached). Immediately preceding the formation of this hinge irreversible deflections do not exceed 0.01 of the span and irreversible losses of load-bearing capacity and rigidity remain within allowable limits. Prestressed elements (also simply supported and reinforced with steel of 15,000 to 18,000 kg per sq cm) withstand heating of the steel up to 200 or 300 C; above 300 C the irreversible losses of load-bearing capacity begin to increase rapidly while even at temperatures only slightly above 200 C the irreversible losses of rigidity exceed the allowable limit (fixed at 20 percent). For cold drawn wire at 8000 to 12,000 kg per sq cm, maximum allowable heating should be determined by additional tests. In conventionally reinforced elements, restraint at supports leads to an increase in limit resistance as long as the ratio of the effective steel cross sections at the supports and in the span is not less than 2. In elements prestressed with high strength steel wire, restraint at the supports does not produce an increase in limit resistance.

General

Proceedings of Concrete Convention 1959, German Concrete Association (Vorträge Betontag 1959, Deutscher Beton Verein)

Druckerei Chmielorz, Wiesbaden, Herrnmühlgasse 11A, 456 pp.
Reviewed by H. H. Werner

The first concrete convention since the war convened in Munich for 3 days and presented a great review of the development and state of concrete in Germany. The first part, as usual, was devoted to speeches, historic reviews, developments, summaries, prices, etc. The Russian delegation presented a paper on the development and state of their concrete industry, particularly the importance of their precast concrete. Extensive German reconstruction with concrete is discussed and illustrated. Following are the highlights of the most interesting papers presented at this convention.

Shell structure of the Exhibition Palace in Paris (in German)

A film and explanation presented this three-point supported triangular shell structure of 740 ft side length. Preliminary studies, design analysis, stability and buckling investigations, stress and strain influences are well presented. Construction methods, foundations and anchorages are illustrated and described.

Suspended roof of the training and exhibition hall in Dortmund, Westphalia (in German)
Presented by F. Vaessen

This prestressed post-tensioned cable suspended roof covers 260-360 ft clear area. The roof suspension cables, span-

ning the short way, are carried over outwardly inclined columns down through diagonal backstays. A prestressed girder in a 36 ft deep tunnel provides counterweight anchorage. The roof cables are arranged in a slightly saddle shaped form and are anchored and prestressed in a margin slab. The cables pass through precast concrete ribs, scaffold supported before tensioning. Gypsum-pumice planks span between ribs and grouting, insulation and waterproofing complete the roof. The connection between the margin slab and the cable suspended cable roof is kept flexible by bituminous cone encasing and lead bearings. Wind forces were established on model tests. This interesting roof is well described and illustrated.

Work of the European concrete Committee (CEB) (in German)
Presented by E. BORNEMANN

A summary of the foundation, purpose, program, etc., of this important committee. In its first 6 years of existence great strides have been made in pooling knowledge and facilities. Subcommittees are working hard at coordinating and defining various fields and progress has been made, and is being made constantly, in a most satisfying and cooperative manner.

Ready-mix and transit-mix in Germany—Its development and prospects (in German)
Presented by E. E. DOMSCHEIT

This method of concrete supply was started in Germany only in 1953 and at this writing (1959) has developed only 20 plants. Its possibilities and limitations are discussed and a great future is envisioned.

Outstanding highway bridges (in German)
Presented by W. KLINGENBERG

A summary of recent concrete bridges with general considerations in planning, adaption, etc., and fine illustrations. The great adaptability of concrete to solve many problems in bridge construction is shown on a variety of projects, as a good testimony for German concrete engineering.

The New Mangfall Bridge (in German)
Presented by U. FINSTERWALDER

A parallel chord lattice truss bridge in prestressed concrete of three continuous, 295, 330, 295-ft spans, built on the existing tall concrete piers of a wardamaged steel bridge. This is an outstanding example of concrete in a field, even in Germany, normally reserved for steel. Cantilevered carriage form was employed, progressing from one end and using temporary intermediate supports. Each panel was installed and prestressed in two separate operations, reusing the form 48 times. The upper chords carry four traffic lanes and the lower chords carry two light local traffic lanes.

Foundation pit enclosure for the new radio and television studio of Munich (in German)
Presented by A. KIRCHKNOPF

A clear unobstructed 2½ story deep dry pit without noise or soil disturbance was constructed with drilled-in caissons (French Benoto system). Alternate caisson steel cylinders were drilled down, excavated and filled with concrete as cylinders are withdrawn. This first series is spaced slightly closer than the caisson diameter and the next series of caissons slices off the overlapping part of the preceeding concrete and forms a solid concrete wall on filling. These last caissons receive reinforcing baskets. A reinforced concrete waler is tied back with prestressed anchors to various types of deadmen.

Construction with big floating reinforced concrete boxes (in German)
Presented by E. SCHORN

A general review of applications, requirements, design, construction, launching and placing of such floating boxes is given. Many examples are well-illustrated and described with particular attention to the vehicular tunnel under the shipping canal at Rendsburg. The center piece of this

four lane 460 ft double tube tunnel was installed in one unit.

Up-dating the German concrete code (in German)

This gives an inside view on the situation of the German code, where a general revision is in process. A number of committees have started to get the code on a new wide basis. Completely new concepts and new fields are to be incorporated, yet bearing basic concepts and the average job conditions in mind.

Reports by several committees and discussions followed:

(a) Reinforcing steel—New classifications as to grades, sizes, formation types, etc., are required with less variations and more definite specifications.

(b) Design and construction conditions—Protective concrete cover, spacing of reinforcing, construction and contraction joints, minimum sizes and over-all stiffness are chapters to be re-evaluated.

(c) Compression and buckling—Review of slender column and wall design is suggested in view of weakening creep influence.

(d) Reinforcing for bending, bending with axial forces, diagonal tension and for torsion—For diagonal tension and torsion clarification and investigations are required at the support areas, for various deformed bars, at moment zero points of continuous beams and for stub beams or brackets. Clarifications are required in bending reinforcing for new high strength concretes, high strength steels, deformed bars, wall beams, etc., and all this with special attention to ultimate strength design.

(e) Slabs, flat slabs, and flat plates —New concepts are to be incorporated with special attention to deflections and edge conditions. Flat plates, still quite new in Germany, are to be investigated for inclusion in the code.

(f) Walls, folded plates and shells—Provision for these is to be incorporated into the new code. This complex chapter still requires a lot of work to arrive at satisfactory definition. It may be handled in a general outline and under design conditions.

New methods regarding the investigation of membrane structures (in German)
Presented by W. TEDPE

Strain investigations by optical reading instruments are presented. Starting with tests using polarized light passing through translucent models, a new method is introduced, using reflectorized investigation from one side only. Strain and angle variation are measured through a microscope using polarized light reflected from mirrorized spots on the outside and/or inside of model. Direct strains and angular variations can be measured with a great degree of precision. Comparative tests on cylinder shells and skew bridges were given to establish the veracity of this method.

Slabs supported on two sides over corner (in German)
Presented by A. MEHMEL

An analytical investigation of various two-way corner slabs on various supports, starting with the corner cantilever, through the cantilever around a corner, to the Greek cross shape slab. A coordinate point system was analyzed with simultaneous linear equations by use of an electronic computer (IBM 650). The results were compared with earlier investigations and illustrated and described. Two cases were completely worked out and reinforcing suggestions given.

Initial and long-time deflections of cracked-state reinforced concrete beams (in German)
Presented by F. LEONHARDT

Deflection becomes a problem with increased concrete and steel stress and more slender members. Deflection limitations by the code are proposed, based on long-time shrinkage and creep and related to steel stresses and percentage to concrete stresses and to slenderness. Tables and charts of limiting slenderness ratios for various conditions are given. Deflection design

formulas and simplifications of same are presented.

Regarding a new theory of bending (in German)
Presented by H. RUSCH

The proposed changeover from elastic to plastic design has presented a number of questions in the physical assumptions. Fair agreement between assumptions and tests exists only for lightly reinforced bending members, while heavily reinforced and/or axially loaded members and long-time effects show considerable deviations. Numerous tests on prismatic compression "blocks" with various eccentric loadings and time, stress and strain controlled loadings showed new strain and breakpoint relations. Attempts are made to correlate these findings with actual loading conditions and to formulate a new bending theory. The idea is to find a theory that truly corresponds to all the factors involved and to formulate design criteria which give a true safety factor.

Conversion factors and tables

O. T. ZIMMERMAN and IRVIN LAVINE, Industrial Research Service, Dover, N.H., 3rd Edition, 1961, 680 pp., $7.50; $8.25 outside U.S.

A comprehensive collection of over 15,000 conversion factors useful to technical and scientific workers. Unusually thorough in its coverage of units of measure peculiar to countries round the world.

Reservoirs and tanks of reinforced concrete (Cuves et réservoirs en béton armé)

L. P. BRICE, *et al.*, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 146, Feb. 1960, pp. 227-285
Reviewed by HENRI PERRIN

This symposium groups:

(a) Specifications drawn up by the French concrete contractors association

(b) French translations of ACI Specifications and Portland Cement Association bulletins for rectangular tanks.

(c) Tables and diagrams for rectangular plates with triangular loadings

(d) French translations of PCA bulletins for cylindrical tanks.

Frank Lloyd Wright: Writings and buildings

EDGAR KAUFMANN and BEN RAEBURN (Editors), Horizon Press, Inc., New York, 1960, 346 pp., $3.95

A survey of the work of Frank Lloyd Wright presented in his writings, complimented by more than 150 illustrations, from the early 1890's to 1959. In addition the book includes a comprehensive list of Wright's executed buildings now standing. Topics discussed include Wright's concrete masonry houses as well as other concrete projects either designed or built.

Engineering drawing and geometry

RANDOLPH P. HOELSCHER and CLIFFORD H. SPRINGER, John Wiley & Sons, Inc., New York, 1961, 2nd Edition, 612 pp., $8.95

A clearly written and generously illustrated textbook intended to cover basic drawing courses, fundamental descriptive geometry, advanced courses in the professional fields, and fundamental work in graphic computation. The presentation adheres rigorously to third quadrant projection, not only for the three principal views but for auxiliary views as well. The book is designed for engineers, with emphasis on principles rather than manual skills.

Reinforced concrete in architecture

E. D. MILLS, *Reinforced Concrete Review* (London), V. 4, No. 6, 1957, pp. 383-391 and 397-402; discussion pp. 391-396
Reviewed by WILLIAM R. LORMAN

A fine exposition of the past, present, and future of concrete. Delineates the architectural guide lines necessary for successful application of reinforced concrete to future structures (box frame as well as curved shell). Surface finishes, workmanship, reinforcement coverage, and design potentialities conclude the article. Contains 12 photographic illustrations which exemplify the author's thesis. The discussion by nine experts deals mostly with problems involving concrete exteriors. The companion to this paper is Nervi's "Concrete and Structural Form" which is also reviewed in this issue on p. 672.

Underground structures

Consulting Engineer, V. 15, No. 6, Dec. 1960, pp. 98-127

Reviewed by ARON L. MIRSKY

The whys and wherefores, and the current state of the art.

Reinforced concrete

JOHN FABER and FRANK MEAD, D. Van Nostrand Co., Inc., New York, 1961, 532 pp., $14.75

A revised edition of Oscar Faber's original book of this title. Design procedures are based on British standard codes of practice. The 24-page chapter on properties of reinforced concrete is particularly interesting because it is in somewhat more detail than in most works of this kind.

New chapters have been added on slabs, retaining walls, and concrete roads. The chapters on elements of design, foundations, bunkers and silos, shell roofs, water towers and reservoirs, chimneys, and prestressed concrete have been enlarged. Both the elastic theory and plastic theory are discussed in the design chapter.

Concrete and structural form

P. L. NERVI, *Reinforced Concrete Review* (London), V. 4, No. 4, 1956, pp. 251-264

Reviewed by WILLIAM R. LORMAN

The author reveals the current Italian trend toward uniting design and erection of concrete structures and explains the manner in which such work is contracted. He points out why a design that is efficient, aesthetically and technically, must of necessity be the most economical. He describes and illustrates stadiums, hangars, and exhibition halls all of which (judging by the eight photographs) portend the future and certainly deserve at least as much attention as Candela's contributions to architectural concrete. Nervi concludes this brief presentation with his excellent views concerning the future of the link between reinforced concrete and architecture; the need for international collaboration in this direction is emphasized. For additional information your reviewer refers the interested reader to E. D. Mill's paper "Reinforced Concrete in Architecture" which is reviewed separately.

Fire rating of structural materials, structural units, plaster, and coatings (in Swedish)

Statens Provningsanstalt, Stockholm, Meddelande 66, 1960, 19 pp.

Reviewed by MARGARET CORBIN

Pamphlet summarizes information on fire rating and fire tests based on the recommendations of the Swedish Statens Provningsanstalt (Official Testing Laboratory). Contents include: terms and definitions; regulations for fire rating and fire tests; recommendations for a classification of building materials and structural units on the basis of fire resistance. The standard time-temperature curve to be followed in fire tests is given: 540 C at 5 min; 705 C at 10 min; 845 C at 30 min; 925 C at 1 hr; 1010 C at 2 hr; 1120 C at 4 hr; 1260 C at 8 hr.

Giant cement mill in the USSR (Une cimenterie géante en URSS)

Revue des Matériaux (Paris), No. 543, Dec. 1960, pp. 343-353

Reviewed by PHILLIP L. MELVILLE

A cement mill, claimed to be the largest in the world with a daily production of 5700 metric tons, is under construction at Atchinsk in Siberia. Two wet process rotary kilns will produce 2500 tons of clinker each. The plant was designed and manufactured in France and shipped by sea, rail, barge, and sled to the site. Raw materials are limestone and by-products from the extraction of aluminum from nephelinite at a nearby industrial complex. The kilns will be 175 m long and 6 m wide fueled with powered lignite. All equipment is designed to function at from −40 C to +35 C. Storage in 16 silos will hold 64,000 metric tons feeding bulk rail shipping and bags to be loaded on trucks. Power requirements for the plants are 40,000 kw and production is estimated with automation at 1600 tons per man year.

NOVEMBER 1961

NEWS LETTER

memo from the **President:**

Concrete Can Be Better in Winter

Temperatures, if not the snowflakes, are falling these days on many a concrete construction job. At this time of year we exchange some of the difficulties with hot concrete during summer for the necessity of keeping it from freezing in winter. Actually, however, with relatively simple precautions in cold weather, circumstances are considerably more favorable than in warm weather for development of the potential qualities of concrete.

Concrete placed and cured at relatively low temperatures safely above freezing will have better ultimate strength, less likelihood of plastic shrinkage, and reduced drying and thermal shrinkage. Moreover, its chances of being better cured are improved because moisture loss is more easily prevented. Despite the recommended 1 percent of calcium chloride to assure sufficient maturity to withstand weathering at the end of minimum practical periods of protection, the lower initial and curing temperatures are favorable to later development of better concrete quality. Clearest among contributing factors is the lower amount of mixing water required in cooler concrete as mixed. Less clearly understood but unmistakable is the superiority of cement gel formed and hardened more slowly at lower temperatures.

For these reasons the opportunity that exists to place cold concrete safely in cold weather should not be overlooked. Some jobs pay a significant premium to do this in warm weather and it is unfortunate when this opportunity is lost in cold weather through mixing at uselessly high temperatures in the mistaken belief that this will somehow protect the concrete later from freezing.

This is not the place for a discussion of protection methods but it does seem worthwhile to record a reminder of the great and automatic advantages of proper insulation for this purpose for unformed concrete and where forms do not have to be reused before the required period of pro-

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tection has elapsed. This method has been used with proven effectiveness in even the coldest weather in which concrete is likely to be placed. Its effectiveness is not so much through keeping the cold out but through conserving the heat of hydration which is slowly generating in the hardening and maturing concrete. Insulation will not be effective unless it is maintained tightly against the forms or the concrete. As with any method of protection, enough temperature measurements of the concrete, especially at corners and edges should be taken to confirm that sufficient insulation has been used to maintain the required minimum temperatures.

Anyone concerned with concrete construction this winter will find it worthwhile to review "Recommended Practice for Winter Concreting (ACI 604-56)" and follow its suggestions. It was published in the ACI JOURNAL for June 1956, p. 1025. Informative related material will be found in its appendix and in its references, and in the ACI JOURNAL for March, 1956, p. 803, and March, 1957, p. 905.

With such a renewed understanding of proper practices and their advantages, ACI members can look forward to a winter of good concrete work free of the needless failures and impairments of quality which happen when these procedures are not understood—or required by informed inspection.

Lewis H. Tuthill
President

ASEE honors Newmark

Nathan M. Newmark, head, Department of Civil Engineering, University of Illinois, Urbana, was recently honored by the American Society for Engineering Education for his outstanding research contributions as an engineering educator.

Professor Newmark was presented the 1961 Vincent Bendix Award at the annual meeting of ASEE held at the University of Kentucky, Lexington. He received a gold medal and citation praising him for "outstanding contributions in research" and for "leadership in developing a great research facility."

Professor Newmark has been at Illinois since 1930, has been in charge of the structural research laboratory since 1946, and civil engineering department head since 1957. Under his leadership the laboratory has attained international recognition. He is currently a member of ACI Committee 318, Standard Building Code.

'62 ACI Nominees

President--Raymond C. Reese
Vice-President--Bryant Mather

The Institute Committee on Nominations, headed by Delmar L. Bloem, associate director of engineering, National Sand and Gravel Association and National Ready-Mixed Concrete Association, Washington, D.C., has announced candidates to be submitted to member ballot. Those elected from these nominees will take office at the 58th annual convention, March 12-15 at the Brown-Palace Hotel, Denver.

For President

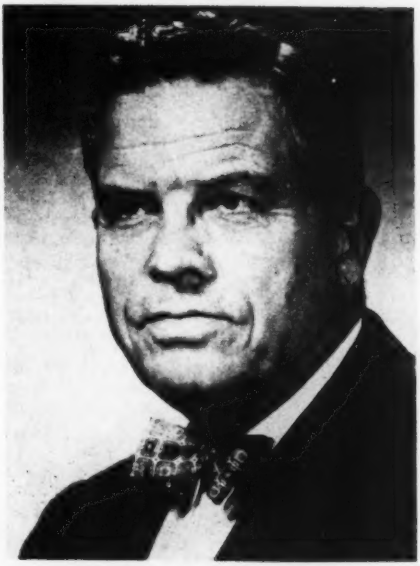
Raymond C. Reese, Raymond C. Reese Associates, Consulting Engineers, Toledo, Ohio, has been nominated to succeed Lewis H. Tuthill as president for a 1-year term beginning at the March 1962 convention. He is now completing his second year as Institute vice-president.

An ACI member since 1936, Mr. Reese has devoted much time to ACI, participating in nearly every phase of Institute activity. He is currently a member of Committee 208, Bond Stress; Committee 315, Detailing Reinforced Concrete; ACI-ASCE Committee 326, Shear and Diagonal Tension; and Committee 401, Specifications for Structural Concrete.

He has served with distinction as chairman of two committees: Committee 315 under his direction from 1948 to 1957 developed two editions of the detailing manual, the highway detailing manual, and the present 1957 edition which combines these two important and widely accepted standards. He is currently chairman of Committee 318, Standard Building Code, which is now completing the time-consuming task of revising the ACI Building Code.

Mr. Reese has served on the Standards Committee and the Building Committee and is currently a member of the TAC Committee on Monographs. He is an ACI representative on the ACI-CEB Collaboration Committee and the Reinforcing Concrete Research Council.

In 1957 Mr. Reese represented ACI at two important conferences in Europe: RILEM Symposium, Stockholm, Sweden, June 1957 and Second Symposium on Concrete Shell Roof Construction, Oslo, Norway, June 1957. While in Europe he met with members



Raymond C. Reese

Detroit Edison's Newest Turbine Generator Has a Fly Ash Concrete Base

The 325,000 KW turbine generator being installed on the base pictured above will subject the two-story, monolithic structure to constant mechanical and electrical vibration when in service.

Structurally sound concrete is vitally needed here to absorb the energy from these vibrations and assure perfect alignment for operation of the generator. Fly Ash, correctly proportioned in the mix, made a real contribution to this requirement by minimizing voids and adding strength to the concrete as its pozzolanic action continues, indefinitely.

The close spacing of rods in the 190 tons of reinforcing steel used here called for concrete of high workability. It was also essential to produce a final structure that would undergo the smallest possible changes in volume. Both objectives were achieved with the use of Fly Ash in the concrete. The required 28-day compressive strength of 4,000 psi was readily surpassed; test results below:

No. of 6" dia. x 12" Specimens	Age, Days	Average Comp. Strength, psi
6	14	4,180
12	28	5,080
11	90	6,290
9	360	7,200

This is the thirteenth consecutive turbine generator base utilizing Fly Ash concrete built for this company. Why not make Fly Ash a factor in your next job?

Each of the companies below has technical data and competent engineers to help you in designing the most effective mixes employing cement and Fly Ash. We invite your call for more specific information.

McNEIL BROTHERS, INC.
P. O. Box 72, Devon Station, Milford, Conn.

CHICAGO FLY ASH COMPANY
228 N. La Salle St., Chicago 1, Ill.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

WEST PENN POWER COMPANY
Cabin Hill, Greensburg, Pa.

WALTER N. HANDY COMPANY, INC.
1846 E. Sunshine St., Springfield, Mo.

of the Comité Européen du Béton, an international committee of engineers striving for better coordination and unanimity of action in matters relating to concrete. From this has grown an increasing affinity between CEB and ACI. Mr. Reese attended a CEB meeting in Vienna, Austria, in 1959.

Mr. Reese has had six papers published in the ACI JOURNAL, co-authored a textbook *Introduction to Reinforced Concrete Design*, and was author of the *CRSI Design Handbook*. In 1953 he received the CRSI award for "his contributions to reinforced concrete construction through design" and in 1957 the ACI Lindau Award "in recognition of his years of effective coordination of practice and instruction through textbooks and technical papers on reinforced concrete."

Educated in engineering at Massachusetts Institute of Technology and Pierce Hall, Harvard, he spent several years as an industrial engineer with the General Electric Co. Since 1922 he has been in private practice as a consulting engineer.

For Vice-President

Bryant Mather, Chief, Special Investigations Branch, Concrete Division, U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Jackson, Miss., has been nominated for a 2-year term as vice-president.

An Institute member since 1944, Mr. Mather has long been active in ACI technical committee work. He is currently a member of the Institute's Board of Direction. He served three terms as chairman of the ACI Technical Activities Committee and was a member of that committee a total of 10 years. He is also currently serving on Committee 116, Nomenclature; Committee 212, Admixtures; and Committee 612, Recommended Practice for Curing Concrete.

Mr. Mather has authored numerous technical papers and is active in committee work of the Highway Research Board and a number of other professional societies. He is chairman of ASTM Committee C-9, Concrete and Concrete Aggregates, and a member of the U. S. Committee on Large Dams of the International Commission on Large Dams. In 1960 he was elected a Fellow of the American Association for the Advancement of Science.

Bryant Mather

In 1959 he was a recipient of the ASTM Award of Merit and in 1961 was corecipient with Leonard Pepper of the Sanford E. Thompson Award of Committee C-9 for their paper on mineral admixtures.

Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York, was elected to a 2-year term as vice-president at the 1961 convention, and will continue in that office.

Board of Direction

Nominated to the ACI Board of Direction for 3-year terms beginning at the March 1962 convention are: J. D. Lindsay, J. Neils Thompson, William R. Waugh, and Hubert Woods.

J. Douglas Lindsay, engineer of materials, Illinois Division of Highways, Springfield, Ill., has been an ACI member since 1952. He is currently a member of Committee 115, Research, Committee 214, Evaluation of Results of Strength Tests of Concrete, and Committee 325, Structural Design of Concrete Pavements for Highway and Airports, being chairman of the subcommittee on continuously reinforced pavements of the latter committee.

Mr. Lindsay was graduated from the University of Illinois, Urbana, with a BS in mechanical engineering. He joined the Illinois Division of Highways in 1924 and has served continuously in its Bureau of Materials in various capacities since that time. Much of his work has been in research and development. He was responsible for the plan-

J. D. Lindsay

J. Neils Thompson

William R. Waugh

Hubert Woods

ning of the experimental continuously reinforced concrete pavement which was constructed in 1947-48 and directed the research work in connection with its construction and the subsequent performance studies. He was appointed assistant engineer of materials in 1948 and promoted to engineer of materials in 1951.

Mr. Lindsay is active in a number of professional societies. He is chairman of the Operating Committee on Materials of the American Association of State Highway Officials. He is the author and coauthor of a number of technical papers.

J. Neils Thompson, a faculty member at the University of Texas, Austin, since 1941, is now a professor of civil engineering, director of the Balcones Research Center, and also director of two component laboratories of the Research Center, which is a division of the University of Texas.

Professor Thompson has served as a consultant to industry in the field of structural mechanics and is the author of some 70 technical publications, books, papers, and reports in the fields of concrete, structures, and materials. He received the ACI Wason Medal jointly with Past President Phil M. Ferguson in 1953 for noteworthy research in 1953, for their paper, "Diagonal Tension in T-Beams Without Stirrups," and in 1960 was presented the "Engineer of the Year Award—1960," Travis Chapter, Texas Society of Professional Engineers, Austin. He has held research grants from the Army, Navy, Atomic Energy Commission, FHA and various industrial organizations.

Professor Thompson is a former president of both the Texas Society of Professional Engineers and the Travis Chapter of the society. He has been a director of the National Society of Professional Engineers since 1955.

An ACI member since 1943, he is currently a member of Committee 209, Creep and Volume Changes in Concrete, and Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete. Professor Thompson was chairman of Committee 213 from 1956-58.

William R. Waugh, chief, Concrete Branch, Engineering Division, Civil Works, Office, Chief of Engineers, Washington, D.C., has been a member of ACI since 1939. He currently is chairman of Committee 621, Aggregates, and ACI representative on ASA Sectional Committee (Z23), Sieves.

Mr. Waugh received his civil engineering degree from Alabama Polytechnic Institute, Auburn, and following graduation worked for about 3½ years with the Dixie Construction Co. and Allied Engineers, Inc., on construction of dams and hydroelectric power plants for Alabama Power Co. and Georgia Power Co. He was with the Tennessee Valley Authority from 1934 to 1945 on construction of multipurpose river development projects.

He joined the Corps of Engineers in 1945, working 3 years in the Norfolk District on foundations and materials investigations in connection with planning and design of dams. Since 1948 he has been in Office, Chief of

Engineers. He has been chief of the Concrete Branch, Engineering Division, Civil Works since 1954.

Active in numerous professional societies, Mr. Waugh is a member of the U. S. Committee on Large Dams and its Subcommittee on Concrete; Fellow, American Society of Civil Engineers; and a member of the Advisory Committee, Long-Time Study of Cement Performance in Concrete.

Hubert Woods, director of research, Research and Development Division, Portland Cement Association, Chicago, has been an ACI member since 1928.

Mr. Woods was employed by the Riverside Cement Co., Los Angeles, after receiving a BS in chemical engineering from the California Institute of Technology in 1923. In 1925 he was promoted to the position of chemical engineer and organized the Riverside Cement Co. research department the following year. From 1926-49 he was research director and in 1934 took on the additional duties of chief chemist. In 1947 Mr. Woods became director of research for PCA.

He is the author of numerous technical papers on cement and concrete and cement manufacture. He is currently chairman of Committee 201, Durability of Concrete in Service, and a member of the executive group of ACI Committee 115, Research.

Mr. Woods is a member of the American Ceramic Society, the Highway Research Board, and the American Society for Testing and Materials.

Nominating Committee

Members of the Nominating Committee in addition to chairman Bloem were Harold Allen, Bureau of Public Roads, Washington, D.C.; Phil M. Ferguson, University of Texas, Austin; Joe W. Kelly, University of California, Berkeley; Thomas B. Kennedy, Waterways Experiment Station, Jackson, Miss., Clyde E. Kesler, University of Illinois, Urbana; Douglas McHenry, Portland Cement Association, Skokie, Ill., and Ivan L. Tyler, Portland Cement Association, Skokie.

Twenty candidates have been chosen for the 1961 Nominating Committee, five of whom will be elected. Candidates are:

E. A. Abdun-Nur
Edward E. Bauer
Irwin A. Benjamin
Boris Bresler
Clayton L. Davis
Frank G. Erskine
Elmo Higginson
Jack R. Janney
Truman R. Jones Jr.
Paul Klieger
George H. Nelson
Gene M. Nordby
Robert E. Philleo
Milos Polivka
Melville E. Prior
Thomas J. Reading
Paul F. Rice
J. J. Shideler
David Watstein
Cedric Willson

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Denver, Colo.

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ACI Exhibits Chairman
831 — 14th Street
Denver 2, Colorado

Winter concreting at Junction Dam

With winter nearly here, concrete men have begun to rearrange their construction programs to take advantage of as much of the remaining construction season as possible. Growing advancement in our knowledge, however, makes continued concreting through the winter months less dangerous. One job that was not stopped because of the detrimental weather was the Junction Dam in the Sierra Nevada Mountains near Placerville, Calif., 50 miles east of Sacramento.

This 165 ft high thin arch structure, part of a comprehensive development

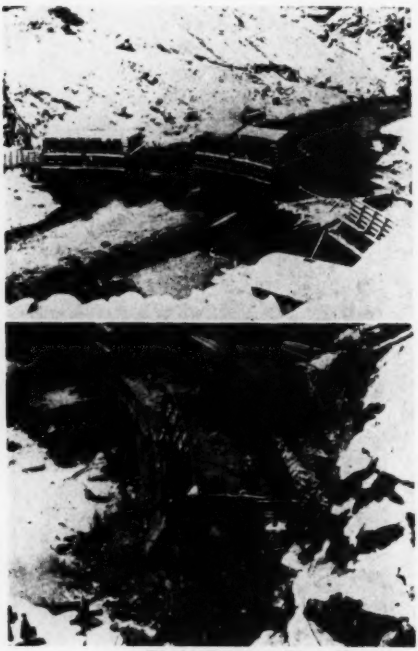


Photo courtesy Blaw-Knox Co.

to provide hydroelectric power for the Sacramento Municipal Utility District, was on such a tight construction schedule that the contractor had to continue concreting through some of last year's adverse winter weather. Steel cantilever forms, insulated with rock wool were used to keep the job on schedule. The forms allowed placing of 250 cu yd of concrete each day by providing for 5 ft lifts of the 2½ in. slump concrete. The concrete was also kept warm with gasoline-fired heaters under polyethylene-covered shelters.

Concrete was moved to the site in 2 cu yd buckets on flat bed trucks. A crawler mounted crane was employed to shift buckets to the placing site, and also to help strip and reposition forms.

Contractor was Fruin-Colnon Contracting Co., St. Louis. Bechtel Corp., San Francisco, supervised the construction.

Prestressed pavement symposium added to 4th FIP congress

A 1-day symposium on prestressed concrete roads and airfield runways has been added to the program for the 4th FIP (Féderation Internationale Précontrainte) Congress in Rome and Naples, May 27 to June 2, 1962. It will be held on Saturday, June 2 at Naples. Members will have the choice between attending this symposium or going on the technical visits arranged for that day by the Italian organizing committee.

Ferguson appointed to ACI-CEB Committee

Past President Phil M. Ferguson, professor, Department of Civil Engineering, University of Texas, Austin, has been appointed to the ACI-CEB (Comité Européen du Béton) Collaboration Committee.

Eivind Hognestad, manager, Structural Development Section, Portland Cement Association, Skokie, Ill., is U. S. chairman of the committee. Raymond C. Reese, Raymond C. Reese Associates, Consulting Engineers, Toledo, Ohio, and Chester P. Siess, professor, Department of Civil Engineering, University of Illinois, Urbana, were previously appointed to the 4-man committee.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees Included are *new* appointments only.

Committee 201, Durability of Concrete in Service

John C. Sprague
Consulting Concrete Engineer
Marietta, Ga.

Committee 315, Detailing Reinforced Concrete Structures

E. D. Ripple
Long Island Railroad
Jamaica, Long Island, N.Y.

B. K. Thornley, Jr.
Easterby and Mumaw, Inc.
Charlotte, N.C.

Committee 404, Joint Sealants

John M. Arnett
Pavements Branch
Building and Ground Division
Headquarters Air Defense Command
Air Force Base, Colo.

Jack E. Flynn
Pyles Industries, Inc.
Detroit, Mich.

Arthur Hockman
National Bureau of Standards
Washington, D.C.

William H. Kuenning
Portland Cement Association
Skokie, Ill.

George E. LaPalm
University of Detroit
Detroit, Mich.

Remo R. Maneri
Dow Corning Corp.
Midland, Mich.

Bruce R. Schoenfeld
Consulting Engineer
Ross, Calif.

Raymond J. Schutz
Sika Chemical Corp.
Passaic, N.J.

Milton H. Zara
Sonneborn Chemical and Refining Corp.
New York, N.Y.

Committee 613, Recommend Practice for Proportioning Concrete Mixes

A. W. Brust
Washington University
St. Louis, Mo.

Charles A. Vollick
Sika Chemical Corp.
Passaic, N.J.

Committee 716, High Pressure Steam Curing

Cedric Willson
Texas Industries, Inc.
Arlington, Tex.

Committee 717, Practice in Low-Pressure Steam Curing

Paul P. Kraai
Permanente Cement Co.
Permanente, Calif.

Committee 805, Application of Mortar by Pneumatic Pressure

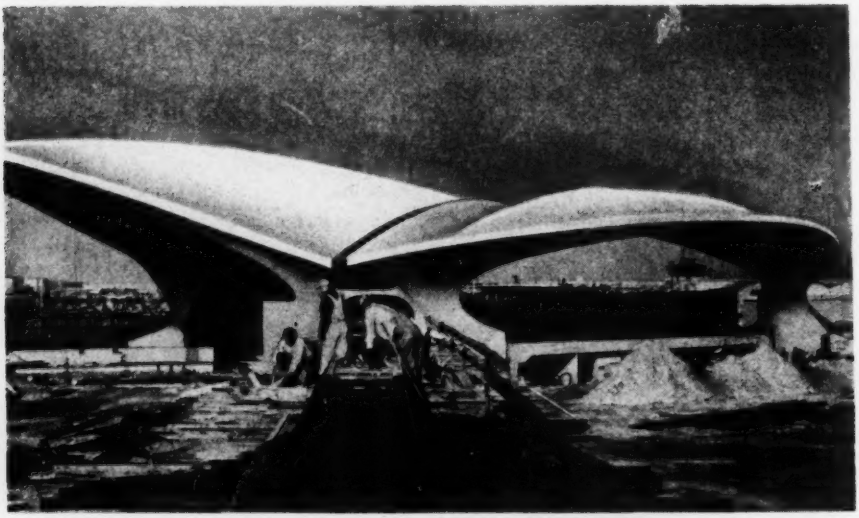
T. G. Clendenning
Ontario Hydro Research Division
Toronto, Ont., Canada

H. A. Cooper
Cooper and Dorris, Inc.
Bartlesville, Okla.

Kuenning to represent ACI on ASA Sectional Committee

William H. Kuenning, senior development engineer, Portland Cement Association, has been appointed to represent ACI on the ASA Sectional Committee on Vermiculite Concrete jointly sponsored by the American Society for Testing and Materials and the Vermiculite Institute.

Mr. Kuenning is also a member of ACI Committee 403, Adhesives for Concrete.



Architect: Eero Saarinen & Associates
Consulting Engineer: Ammann & Whitney
General Contractor: Grove, Shepard, Wilson & Kruge, Inc.

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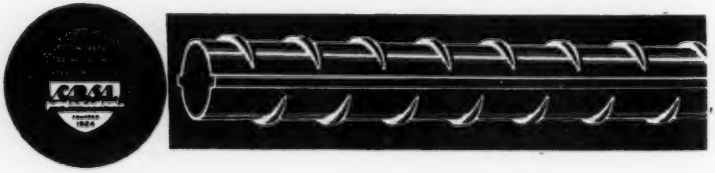
For structures of every type, creative architects know that monolithic reinforced concrete provides greater opportunity for individuality in building design and construction.

Suggestive of a huge bird poised for take-off, the new TWA Terminal Building at Idlewild Airport is a testimonial to the flexibility of this construction method. Its huge concrete shell roof is an arch cantilever design in four continuous monolithic reinforced concrete sections.

On your next project, design with greater freedom — design for monolithic reinforced concrete.

Concrete Reinforcing Steel Institute
38 South Dearborn Street
Chicago 3, Illinois



7-61

West to Denver in '62— ACI 58th Annual Convention

Plans are well underway for ACI's 58th annual convention to be held at the Brown-Palace Hotel, Denver, March 12-15. Past President Walter H. Price, chief research engineer, U. S. Bureau of Reclamation, Denver, heads up the planning committee assisted by many individuals who planned the highly successful meetings held in 1948 and 1954 in Denver.

A special feature of the '62 convention will be an architectural concrete design contest for the students of six Rocky Mountain area colleges and universities, namely, the University of Colorado, the University of New Mexico, Montana State College, the University of Arizona, and Arizona State University. The hypothetical study problem already submitted to the schools consists of an indoor skating rink featuring use of reinforced concrete. Models of the entries will be on display in the convention area, and cash prizes totaling $600 and scrolls will be awarded the winners.

Other features include a buffalo barbecue supper, combined with a tour of the Bureau of Reclamation Engineering Laboratories, which were popular attractions at previous Denver conventions.

Honorary chairmen are Grant Bloodgood, assistant commissioner and chief engineer of the Bureau of Reclamation, and Cris Dobbins, president of the Ideal Cement Co., Fort Collins.

McCarthy elected director of SBEE national council

ACI Director James A. McCarthy, professor of civil engineering at the University of Notre Dame, Notre Dame, Ind., was recently elected a director of the National Council of State Boards of Engineering Examiners at its meeting in Biloxi, Miss. He will serve a 2-year term representing the organization's central zone.

Professor McCarthy has been a member of the Indiana State Board of Registration for Professional Engineers and Land Surveyors for 12 years. He is also a member of the board of appeals for the South Bend, Ind., building code. He is currently serving on ACI Committee 115, Research.

CIB sets dates for '62 session

The International Council for Building Research Studies and Documentation (CIB) will hold its next Congress in Cambridge, England, September 6-11.

The theme of the 1962 Congress will be "The Influence of Changing Requirements and Developments in Materials and Components on Design and Construction."

Additional information is available from Dan R. Hamady, assistant administrator, Office of International Housing, Housing and Home Finance Agency, Washington 25, D.C.

Holcomb named dean of SEATO Graduate School of Engineering in Bangkok

Robert H. Holcomb, professor at the Texas Agricultural and Mechanical College, College Station, Tex., has been appointed dean of the SEATO Graduate School of Engineering in Bangkok, Thailand, to succeed T. H. Evans, who has returned to Colorado State University, Fort Collins, where he is dean of engineering.

The school project, held on the campus of Chulalongkorn University, Bangkok, is a cooperative effort of Australia, France, New Zealand, Pakistan, Philippines, Thailand, United Kingdom, United States, and SEATO, who provide financial support, teaching and administrative personnel.

The United States' contribution to the Graduate School includes nine staff members, major budget support, exclusive support of extension on special training program, and technical periodicals.

Old ACI *Proceedings* volumes available

Arrangements have been made by the Institute with University Microfilms, Inc., Ann Arbor, Mich., whereby ACI members and JOURNAL subscribers may obtain all *Proceedings* volumes (V. 1-56) on microfilm or xerographic reproductions. This arrangement makes available the back volumes that have been out of print for a number of years and will enable many members and subscribers to fill out their libraries of ACI *Proceedings*.

The microfilmed volumes will be provided in film strips of standard dimension each containing a complete volume, including indexes and errata. The xerographic reproductions are available as standard page-size reproductions and can be supplied either by single JOURNAL issues for volumes subsequent to *Proceedings* V. 26, or as complete volumes. Because there was no JOURNAL prior to V. 26, these early *Proceedings*, can be supplied as complete volumes only. Requests for any ACI *Proceedings* material will always be supplied in original printed copies when available; the xerographic copies will only be provided for out-of-print material.

Microfilm copies of *Proceedings* V. 49-56 are available only to ACI members and subscribers.

Information concerning specific material and prices is available from Institute headquarters.

Schutz named head of BRI Committee on Adhesives and Sealants in Buildings

Raymond J. Schutz, vice-president, Research and Development, Sika Chemical Corp., Passaic, N.J., was appointed chairman of the Building Research Institute's 1961-62 Planning Committee on Adhesives and Sealants in Buildings. He has been a member of this committee since 1959.

Long active in ACI committee work, Mr. Schutz is currently a member of ACI Committee 717, Practice in Low-Pressure Steam Curing, and Committee 805, Application of Mortar by Pneumatic Pressure. He is also active in committee work of the American Society for Testing and Materials, and the Highway Research Board.

Mr. Schutz is a full member of RILEM (International Union of Testing and Research Laboratories for Materials and Structures), a member of the National Association of Corrosion Engineers, and the Society of American Military Engineers.

Philadelphia pays tribute to Clair

Philadelphia awarded a Distinguished Citizen Tribute to Miles N. Clair, president, The Thompson & Lichtner Co., Inc., Brookline, Mass., at a special ceremony held in the mayor's reception room at Philadelphia's city hall on September 18.

The tribute credits Dr. Clair, more than any other engineer, with developing the use of precast and premixed concrete. His company pioneered in developing ready-mixed concrete procedures, permitting trucks to supply concrete to projects in large quantities. Under his direction, his company originated many precasting methods.

Dr. Clair's personal research, writings, and consultation service have contributed greatly to the advancement of concrete construction.

He is currently president of the American Society for Testing and Materials. He is past president of the New England section, American Society of Civil Engineers; past president of the Boston Society of Civil Engineers; past director of ACI which he represents on ASA Sectional Committee A-1; and a member of ASA Committee on Reinforced Gypsum. He is a recipient of the Clemens Herschel Award presented by the Boston Society of Civil Engineers. Dr. Clair is also active in many civic organizations.

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Narramore elected NYSAE president

Marve Narramore, managing director of the Perlite Institute, Inc., New York, was recently elected president of the New York Society of Association Executives, representing more than 400 trade association executives in metropolitan New York.

Davis elected to NRC executive committee

Harmer E. Davis, director of the Institute of Transportation and Traffic Engineering, University of California, Berkeley, has been elected a member of the executive committee of the Division of Engineering and Industrial Research, National Research Council, National Academy of Sciences, Washington, D.C.

Mr. Davis, a former member of the ACI Board of Direction, has long been active in ACI technical committee work. He has contributed a number of papers for publication in the JOURNAL and is a Wason medalist.

Bromilow named dean of engineering at NMSU

Frank Bromilow, after several years as head of the civil engineering department at New Mexico State University, has been named dean of engineering and director of the Engineering Experiment Station at NMSU.

An instructor and later assistant professor at the University of Pittsburgh from 1937 to 1946, he was subsequently chief engineer for the Plasteel Products from 1946 to 1948, and a member of the engineering faculty at the University of Florida from 1948 to 1951.

Professor Bromilow will be succeeded as department head by John W. Clark, professor of civil engineering and a NMSU faculty member since 1953.

California Highway Conference scheduled

The 14th California Street and Highway Conference will be held on the Los Angeles campus, University of California, Jan. 25-27, 1962.

The conference is presented annually by the Institute of Transportation and Traffic Engineering, University of California, Berkeley and Los Angeles. E. T. Telford, California assistant State highway engineer, is conference chairman.

BRI Fall Conferences scheduled, November 28-30

Seven timely topics, each of vital interest to the entire building industry complex, make up the program for the Building Research Institute's 1961 Fall Conferences, taking place this year at the Mayflower Hotel, Washington, D.C., November 28-30. The several segments of the conference will be devoted to new factors of environment, design of protected areas, human factors in design, design of a nuclear city, and a comprehensive panel discussion on ways of implementing the new design.

Two ACI members are scheduled to speak in a conference on "Design for the Nuclear Age." Lyndon Welch, chief structural engineer, Eberle M. Smith Associates, Inc., Detroit, will discuss architectural design as related to protection for areas in normal usage, special protection areas for personnel, and design of areas in which essential facilities would be operated during an emergency. Gifford H. Albright, associate professor of architectural engineering and director of the Shelter Research and Study Program at Pennsylvania State University will speak on structural design with the same frame of reference.

A unit of the Division of Engineering and Industrial Research of the National Academy of Sciences-National Research Council, BRI makes its conferences open to the interested public as well as to BRI members and their guests.

"Ready" in a hurry

Control tower on the left controls batching for three ready-mixed concrete batching plants

Photo courtesy Noble Co.

■ READY-MIXED CONCRETE batching has been speeded up with an ultramodern control arrangement by one producer in San Jose, Calif. Borcher Bros. found that 4½ min was too long for loading out a mixer truck and still produce 1000 cu yd of concrete per day. Installation of an automated master batching console reduced loading time to 2 min.

One operator controls separate, simultaneous batching of aggregates, cement, and water for two plants plus batching dry materials only for a third plant. The operator, while remote from operations, has visual contact with the three plants, yard area, and aggregate handling facilities. The tower is air conditioned and connected by an intercommunications system to truck loading stations and dispatcher.

The console used in this operation has provision for remotely batching 10 preset formulas of seven different materials and five admixtures. Moisture content of the sand is compensated for electronically.

PCI holds 7th annual convention in Denver

The Prestressed Concrete Institute, Chicago, held its 7th annual convention at the Brown Palace and Cosmopolitan hotels in Denver, October 15-19.

The keynote address was given by Thomas H. Creighton, editor of *Progressive Architecture*. He spoke on "New Opportunities in Structural Design."

There were five half-day sessions on such subjects as fire resistance, design, connections, research, and outstanding structural projects. The fire resistance session dealt with the philosophy of fire testing, a discussion of ASTM Standard F-119 requirements and reports on test and actual fires in prestressed concrete structures.

The design of prestressed concrete members and structures was discussed in the design session, while multistory and continuous connections, as well as provision for movement, were emphasized at the connections session.

The research session revealed new developments in prestressed concrete. Convention highlights included field trips to outstanding examples of prestressed concrete construction in the Colorado area and several social events.

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Consider the advantages of eliminating the thousands of time-consuming placing and tying operations required by bar reinforcement. It's a lot easier to reinforce concrete with USS American Welded Wire Fabric. We can furnish it in easy-to-handle rolls which are quickly unrolled into position, or in flat sheets sized to your exact specifications. Either way, we believe you can save at least 1/3 placement time. In many cases the building can be finished and ready for occupancy sooner when fabric is used—this is another profitable advantage of American Welded Wire Fabric. ▪ USS American Welded Wire Fabric has many other advantages, too. It is made from cold-drawn, 60,000 psi minimum yield strength wire, meeting ASTM Specifications A-185. Higher working stresses can be used since high bond value is guaranteed by the positive anchorage provided at each welded intersection. Spacing of each reinforcing member is guaranteed to a close tolerance of ± 1/4" and each reinforcing member is cold drawn to a tolerance of .003 of an inch. That means less steel, less freight, lower handling and installing costs. Since it is pre-fabricated, welded wire fabric requires less man hours per ton to place than loose reinforcing members. ▪ All concrete fills or slabs on grade should be reinforced with Welded Wire Fabric because it increases the strength of a plain concrete slab 30%. The cost of the fabric reinforcement slab is therefore much less than an unreinforced slab of equal strength. ▪ For a complete look at the cost and design advantages of USS American Welded Wire Fabric, call or write American Steel and Wire, Dept. 1408, Rockefeller Building, Cleveland 13, Ohio. We will gladly assist you on any concrete reinforcing problem.

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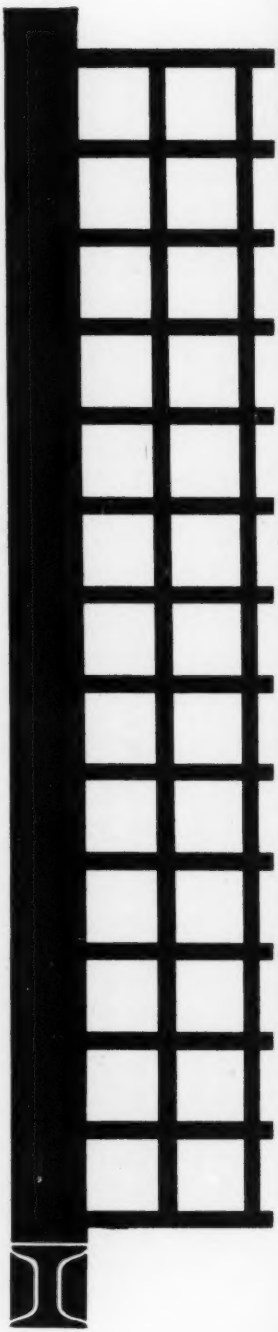
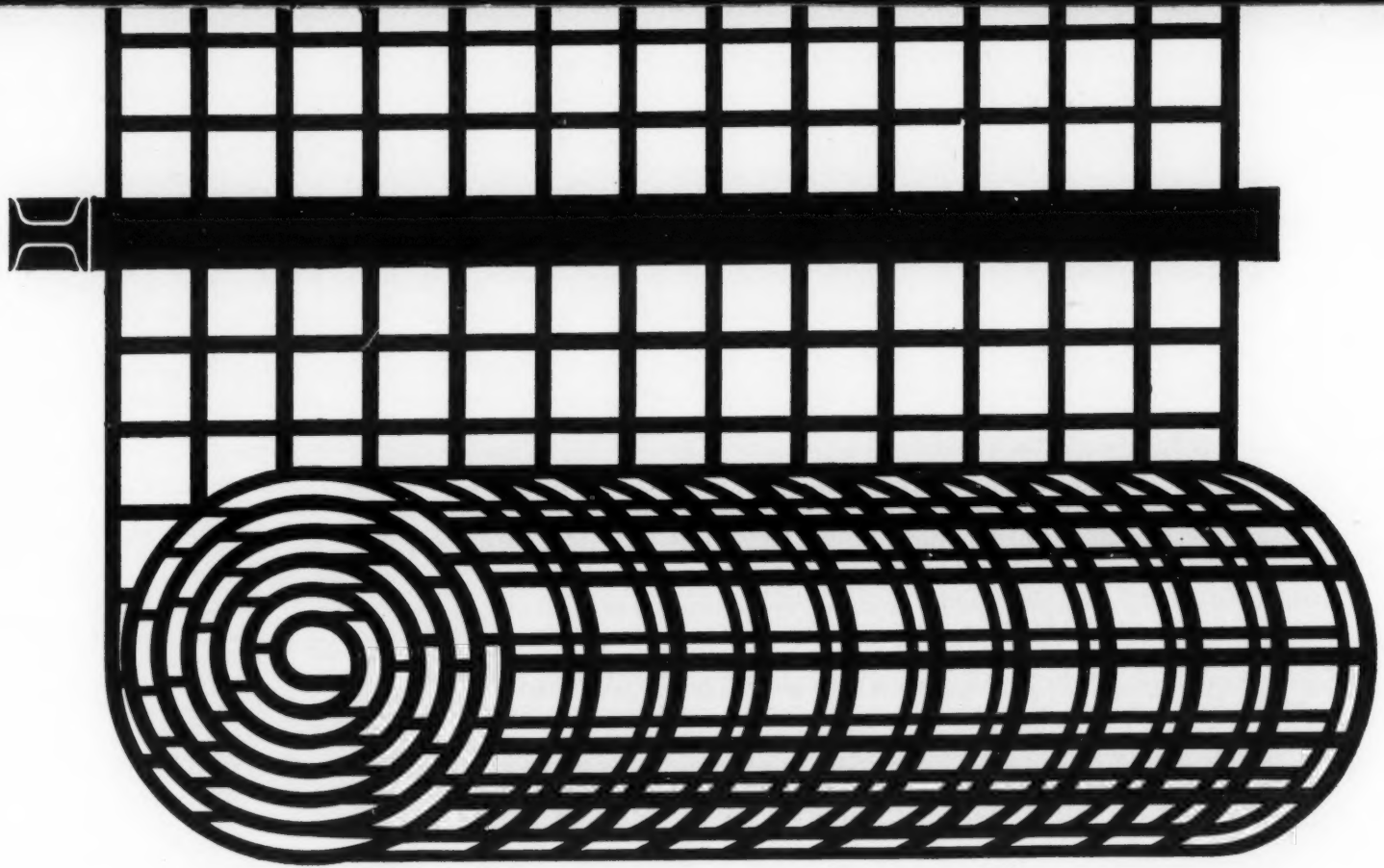




Bower leaves U of I

J. E. Bower, formerly an instructor in the Department of Theoretical and Applied Mechanics, College of Engineering, University of Illinois, Urbana, is now working in the Applied Research Laboratory, U. S. Steel Corp., Monroeville, Pa.

FIP Fourth Congress to be held in Rome

The executive committee of the Federation Internationale de la Precontrainte announces that the Fourth International Congress on Prestressed Concrete will be held in Rome and Naples from May 27 to June 2, 1962.

The final program has been approved with papers selected from 155 technical papers and reports submitted for consideration. Main categories of subjects to be discussed include results of research with special reference to durability and fatigue; site considerations (problems and difficulties as well as remedies and solutions), economics of prestressed concrete in relation to regulations, safety, partial prestressing, and lightweight concrete; and progress in precast factory manufacture and standardization.

A special symposium on prestressed concrete pavements will be held and two sessions will be devoted to outstanding structures in prestressed concrete, one on bridges, viaducts, and elevated roads, and one on buildings and other structures.

T. Y. Lin, professor of civil engineering, University of California, Berkeley, was a member of the program planning committee.

Prestressed Concrete Institute has appointed as national reporters for the sessions describing outstanding structures in prestressed concrete construction, James D. Piper, vice-president, Portland Cement Association, Chicago; C. C. Zollman, partner, Schupack and Zollman, Stamford, Conn., and Professor Lin.

Wiesinger receives PhD at Northwestern University

Frederick P. Wiesinger, associate professor of civil engineering, University of Illinois, Chicago Undergraduate Division, has received his PhD degree at Northwestern University in civil engineering. He was on a leave of absence for 1960-61, while on a National Science Foundation Science Faculty Fellowship.

Dr. Wiesinger came to this country from his native Hungary in 1950 and joined the faculty in 1955. In addition to his research activities, results of which have been published here and abroad, he is also responsible as consulting structural engineer for the design of a large number of buildings and other structures.

Dr. Wiesinger is currently serving on ACI Committee 322, Design of Structural Plain Concrete.

Dundee adds Fleming to Chicago staff

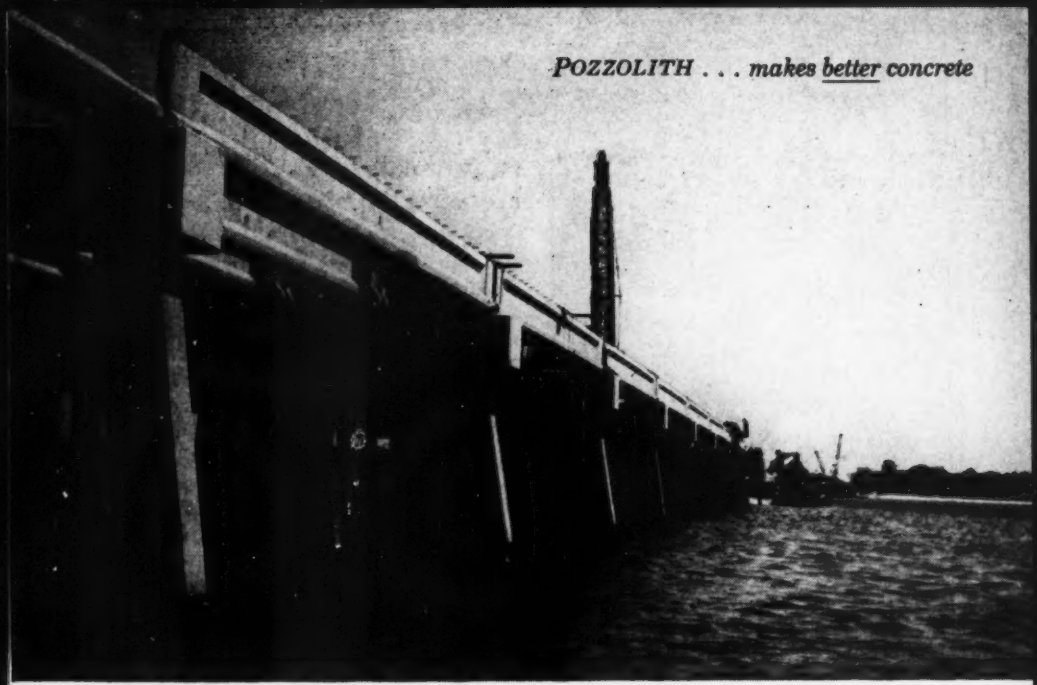
Robert G. Fleming has been appointed a sales representative for Dundee Cement Co., Chicago district.

Mr. Fleming previously served as a highway engineer for the Illinois Division of Highways and as a project engineer for Joyce Brothers Construction Co.

Gray opens consulting office in Mentor

William R. Gray has opened structural and civil engineering offices in Mentor, Ohio.

In 8 previous years as chief structural and civil engineer with McGeorge, Hargett and Hoag, Cleveland, he participated in numerous projects, such as the new 23-story Illuminating Co. building at the Cleveland Public Square and a number of new buildings at Case Institute of Technology, Cleveland.



BILOXI BAY BRIDGE—67′ wide—will connect Biloxi and Ocean Springs, Mississippi with two 26′ clear roadways for 4-lane traffic. POZZOLITH concrete was used throughout the new prestressed structure—including piles, deck beams, deck slab, pile caps and footings. Construction Agency: Mississippi State Highway Dept. • General Contractor: J. B. Michael & Co., Inc., Memphis.

Big bridge over Biloxi Bay

new $6 million prestressed project spans 1 1/2 miles

When traffic starts to roll on the completed Biloxi Bay Bridge in early '62—over 39 miles of prestressed POZZOLITH concrete members will be in place. These consist of 128,000 linear feet of prestressed piles, ranging in length from 45′ to 95′ in four sizes—36″, 30″, 24″ hollow void and 20″ solid, plus 1,512 prestressed deck beams each 52 feet long. Total prestressed and poured-in-place POZZOLITH concrete for the project is 52,000 cubic yards.

A complete on-site prestress yard was set up by the contractor including a 7-lane casting bed and dry batch concrete plant. Three 330-foot lanes are for casting beams, four 465-foot lanes for piles.

Job demands and a tight timetable led to these requirements for the 5000 psi concrete specified: (1) Controlled retardation to assure workability and easy handling for the poured-in-place concrete (footings, pile caps, deck slab) after long transport by barge. (2) High early strength (3500 psi at 36 hours) with low W/C ratio for the prestressed members in order to maintain an extremely high production rate. (Example: 940 feet of piling per day.)

POZZOLITH reduced total water content 12% and provided the *controlled* rate of hardening needed for superior workability under severe time and temperature conditions. POZZOLITH also helped provide greater durability for concrete exposed to sea water corrosion and tested at 28 days up to 8000 psi.

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LOOKING AHEAD

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

Nov. 14-17, 1961—16th Short Course for Industry Technicians, National Sand and Gravel Association and National Ready Mixed Concrete Association, University of Maryland, College Park, Md.

Nov. 28-30, 1961—Fall Conferences, Building Research Institute, Mayflower Hotel, Washington, D.C.

Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Dec. 5-7, 1961—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

Jan. 15-18, 1962—42nd Annual Convention, National Concrete Masonry Association, Americana Hotel, Bal Harbour, Fla.

Feb. 5-9, 1962—32nd Annual Convention and Biennial Show, National Sand and Gravel Association and National Ready Mixed Concrete Association, Conrad Hilton Hotel, Chicago

Feb. 11-15, 1962—Annual Meeting and Exhibition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago

Mar. 12-15, 1962—58th Annual Convention, American Concrete Institute, Brown-Palace Hotel, Denver, Colo.

Adalist announces firm name change

The firm name of North American Inspection and Testing has been changed to Herman Adalist and Associates. The firm has assumed new headquarters on Union Bay Place, Seattle, Wash.

NCMA names new executive assistant

The National Concrete Masonry Association has announced the recent appointment of Robert H. Steel as assistant to executive director, Walter W. Underwood. In this newly created position, Mr. Steel will handle arrangements for NCMA regional meetings and the annual convention and will assist with other phases of the association's regular administrative functions. In addition, he will assume responsibility for an expansion of NCMA activity with regional, state, and local concrete masonry organizations, serving in a liaison capacity.

Mr. Steel comes to NCMA from the John A. Roebling's Sons Division of the Colorado Fuel and Iron Corp., Trenton, N.J. He is a graduate of New Jersey State College, Trenton, and is currently doing post-graduate work at the American University, Washington, D.C.

Errata

The following corrections should be made in "Analysis of Visco-Elastic Behavior of Concrete Structures with Particular Reference to Thermal Stresses," by O. C. Zienkiewicz which appeared in the October 1961 JOURNAL.

p. 383—2nd line from the bottom change "σ_z" to "ε_z."

p. 384—Eq. (4) should read

$$\int_0^t c(\tau,t)\ \frac{d}{dt}\left[\quad\right] dt \equiv C$$

p. 389—3rd paragraph of the Footnote, change

"$\dfrac{1}{E_0}\,\Delta_0 \displaystyle\int_0^t \tau\,\frac{\delta H(t-\tau_0)}{\delta\,\tau}\,d\,\tau$"

to

"$\dfrac{1}{E_0}\,\Delta_0 \displaystyle\int_0^t \tau\,\frac{\partial H(\tau-\tau_0)}{\partial\,\tau}\,d\,\tau$"

p. 390—Bottom line, change "$e^{-0.07t}$" to "$e^{-0.07\tau}$."

Overseas chief named by Lone Star Cement

John C. Mundt has been appointed vice-president in charge of administration of foreign subsidiaries, Lone Star Cement Corp., New York. He was formerly vice-president and manager of the Argentine Portland Cement Co.

Mr. Mundt joined Lone Star in 1956 in Uruguay, and 2 years later became vice-president and manager of the Uruguay Portland Cement Co.

Three firms merge with Associated Sand & Gravel

Everett Paving Co., Everett Development Co., and Skagit Valley Paving Co., all of Everett, Wash., have merged with the Associated Sand & Gravel Co., Everett.

At the present time there will be no change in the ownership, management, and personnel of the companies affected.

Howard F. Sievers and George P. Duecy are managing partners of Associated Sand & Gravel.

Morgan joins Wiener staff

James F. Morgan has joined the J. Wiener Co., Muskegon, Mich., as director of engineering and quality control. All products and activity involve portland cement concrete.

Great Lakes Carbon expands building products operations

Great Lakes Carbon Corp., New York, has formed a new department to market building products.

Establishment of the Perma Products Department and the appointment of Lyle R. Bolster as department sales manager was recently announced by D. L. Marlett, vice-president of the corporation and general manager of its Mining and Mineral Products Division. The newly organized department will be located in the New York City headquarters.

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Walter L. Lowry

Walter L. Lowry, dean of engineering, Clemson College, Clemson, S. C., died September 14 at the age of 54.

Professor Lowry had been a member of the Clemson College faculty since he came here as associate professor of civil engineering in 1949. He became full professor and acting head of the department in 1951. He headed the department from 1952 through 1960 and in 1961 was named dean of the school of engineering.

He was a native of Monroe, N. C., and earned his BS degree in civil engineering from Virginia Military Institute, Lexington. He received his Master's degree in the same field from Rensselaer Polytechnic Institute, Troy, N. Y., in 1939.

Following his graduation, he taught at VMI for 7 years and then was on the RPI faculty for 4 years.

From 1946 through 1949, he was engineer in charge of construction for the Deering, Milliken Co., Inc., for which he supervised the construction of three new mills in South Carolina and the addition to an existing plant in Maine.

In World War II, he rose to the rank of major with the corps of engineers during 3½ years service. Professor Lowry was active in a number of professional societies.

Sabri Sami

Sabri Sami, professor of structural engineering, West Virginia University, Morgantown, was killed in a traffic accident in Stockton, Kan., on September 7.

Dr. Sami, 42, graduated from Cairo University, Cairo, Egypt in 1943. He received his MS and PhD degrees in structural engineering from the University of Illinois, Urbana, in 1951 and 1953, respectively.

Before coming to the United States, Dr. Sami served as a structural engineer in the Egyptian Government specializing in reinforced concrete design and inspection of construction.

He joined the faculty at West Virginia University in 1957. Dr. Sami was employed this past summer at the U. S. Naval Civil Engineering Laboratory at Port Hueneme, Calif.

An ACI member since 1954, he was currently working on three committees: Committee 208, Bond Stress; Committee 314, Rigid Frames for Buildings and Bridges; and Committee 335, Deflection of Concrete Building Structures.

Design of Concrete Pavements

Recommended Practice for Design of Concrete Pavements (ACI 325-58) covers the design of rigid airport and highway pavements and bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Includes recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced concrete. 36 pp. $1.00 per copy, 50¢ to ACI members.

Order from Publications Department, American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Michigan.

Honor Roll

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Jan. 1-Sept. 30, 1961

Have you signed a new ACI member this month? Among your acquaintances working in concrete, there are probably several excellent ACI prospects who would only be too anxious to join if they only knew more about ACI services. Why not tell them about ACI today—and have your name listed in next month's honor roll.

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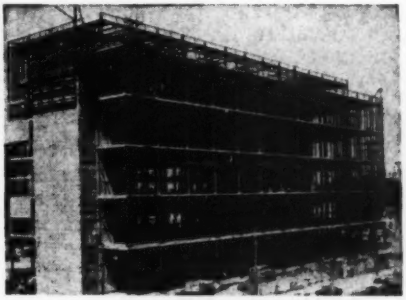

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Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter)_____

For Corporation Membership, ACI representative will be _____

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For our records, please complete both sides of application

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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New Members

The Board of Direction approved applications in the following categories: 62 Individual, 4 Corporation, 10 Junior, and 13 Student, making a total of 89 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,449.

INDIVIDUAL

ANTIA, K. F., Bombay, India (Chf. Exec., M. N. Dastur & Co. Pvt. Ltd.)

BARNES, MARION D., Washington, D. C. (Rsch. Dir., Sulphur Inst.)

BARNETT, WILLIAM O., Dallas, Tex. (Struct. Engr., Thomas E. Stanley, Archs. Engrs.)

BHIDE, M. V., Bombay, India (Asst. Surv. of Wks., Life Ins. Corp. of India)

BOWMAN, WALTER C., Harlingen, Tex. (Cons. Engr., Bowman Swanson Hiester, Archs. Engrs.)

BREWER, ROBERT A., Oakland, Calif. (C. E., San Francisco Bay Toll Crossing)

BROWN, GORDON V., Tucson, Ariz. (Arch.-in-Trng., Arthur T. Brown, Arch.)

BROWN, HARRY E., Charlottesville, Va. (Hwy. Engr., Va. Dept. of Hwys.)

BURLEY, RUSSELL J., Ft. Lauderdale, Fla. (Conc. Engr., Lehigh P. C. Co.)

CARIOTI, BRUNO M., Patrick AFB, Fla. (Lt. CEC, USN)

CAVLI, ADNAN, Ankara, Turkey (Asst. Dir., State Hwy. Dept. Test. Lab.)

CHOPE, FRANK S. J., Tehran, Iran (Constr. Supt., J. A. Jones Constr. Co.)

COIL, HENRY W., JR., Colton, Calif. (Sales Engr., Conc. Conduit Co.)

DE YOUNG, CLARENCE, E., Ames, Iowa (Asst. Matls. Engr., Iowa State Hwy. Comm.)

DIEKMANN, EDWARD F., El Cerrito, Calif. (Struct. Desr., Gilbert, Forsberg, Diekmann, Schmidt)

EL DARWISH, IBRAHIM ALY, Ann Arbor, Mich. (Teacher, Alexandria Univ.)

FABER, J. C., Hong Kong (C. E., S. E. Faber & Son)

FINSTERWALDER, ULRICH, Munchen, Germany (Chf. Engr., Dyckerhoff & Widmann)

GOMEZ JUNCO, ROBERTO, Monterrey, N. L., Mexico (Prof., Instituto Tecnologico)

HARNEY, DAVID R., Torrance, Calif. (Dist. Mgr., Universal Form Clamp Co.)

HAYNES, M. A., Guildford, Surrey, England (Struct. Engr., Cen. Elec. Generating Bd.)

HERNANDEZ P., CARLOS, Bogota, Colombia (Cons. Engr.)

HODGES, WILLIAM S., Muskegon Hts., Mich. (Contr., William S. Hodges Co., Inc.)

JACHONTOFF, IGOR N., Vancouver, B. C., Canada (Des. Engr., H. A. Simons Ltd.)

JOHNSON, CHARLES D., Englewood, Colo. Mgr., Rocky Mt. Prestress Inc.)

KECKONEN, ERNEST V., Birmingham, Mich. (Struct. Engr., McWilliam & Keckonen)

KEMP, JOHN T., Oklahoma City, Okla. (Sales Mgr., Gen. P. C. Co.)

KISHORE JAIN, JAGAT, New Delhi, India (Dep. Irrigation Advr., Min. of Food & Agric.)

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• *By ACI Committee 212*

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

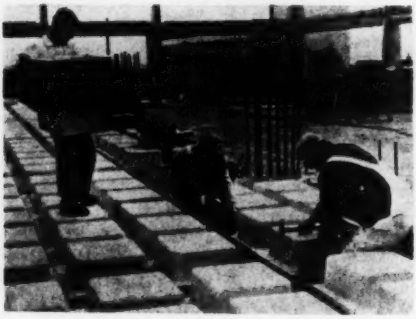
Liquid neoprene coating

Recently developed APCO Liquid Coating is DuPont neoprene rubber in an easy-to-use liquid form, which can be applied by brush, spray, or paint roller. It dries in minutes on concrete, metal, wood, or plastic, forming a continuous rubber film which is said to be impervious to fresh and salt water, oils, acids, and other chemicals.

Permanently flexible, APCO will not peel blister or crack under changing climatic conditions. Manufacturer claims it is ideal for repairing and waterproofing leaking roofs, chimney flashings and pipe joints. APCO can be used in plant maintenance and in skidproofing floors and other surfaces.—Adhesive Products Corp., 1660 Boone Ave., New York 60, N. Y.

Molded fiber glass forming pans

Molded fiber glass forming pans aided in cost reduction on the Pennsylvania State University residence hall project. The forms, 19 x 19 x 8 in., weighed 6¾ lb. Said to be re-usable up to 25 times, they have good



dimensional stability and are light weight, nonwarping, nondenting, nonrusting, smooth, and long wearing with good releasing qualities. The pans used on the Penn State job had a wall thickness of 0.110 in. and were made with a 10-mil veil of fiber glass molded into the surface.

On the Penn State job, 8 to 10 percent of the pans dropped from the surface when the staging was removed. The others were removed in an average 5 sec per sq ft. A "blow hole" in the center of the pans provides easy removal; a blast of compressed air into the hole breaks the bond with the concrete.

The surface resulting from the pan forms is smooth enough to require no additional treatment beyond painting.

A single broad headed nail secures the corner of four pans making setting an easy matter.—Molded Fiber Glass Co., 4401 Benefit Ave., Ashtabula, Ohio.

Scaffolding film

"More Profits for Builders" is the title of a 16-mm sound-color film showing basic scaffolding set-up and how it is used in all kinds of construction and maintenance work across the nation. Many interesting and unusual scaffolding applications are shown including industrial buildings, apartments, aircraft hangars, ship building, churches, theaters, bridges, and reservoirs.

The 20-min film is available on free loan to contractors, builders, engineers, trade groups, universities, and others interested in building and construction.—Thomas J. Barbre Productions, Library Division, 2130 South Bellaire St., Denver 22, Colo.

Chemically resistant concrete

The cement skeleton of cement-containing products shows considerable quantities of numerous lime compounds, including free lime. Even in so-called sulfate cements and in products introduced on the market as being "acid proof," these lime compounds are found. Many of these calcium components, and free lime in particular, are unstable and have little chemical resistance. Therefore, concrete and artificial stone have a marked low resistance to chemicals. Corrosion of concrete in unfavorable surroundings is unavoidable.

Ocrating, the treatment of concrete with silicontetrafluoride gas, gives an improved resistance to the action of aggressive substances and also a valuable increase in strength and impermeability. With such improvement of properties the field of useful application of concrete is considerably widened. Ocrating is a patented process of the Holland firm, Ocrietfabriek N. V.

By ocrating good dense quality concrete special protection becomes unnecessary. The treatment with the fluor containing gas causes a change of all calcium compounds, including the free lime, and chemically resistant and mechanically strong substances are formed. As a contributory reaction a deposit of silicic acid is formed in the micro pores thereby improving the density of the concrete. Therefore, ocrated concrete has a high chemical resistance. At the same time the physical properties are considerably improved.—Ocrietpabriek N. V., Baarn, Holland.

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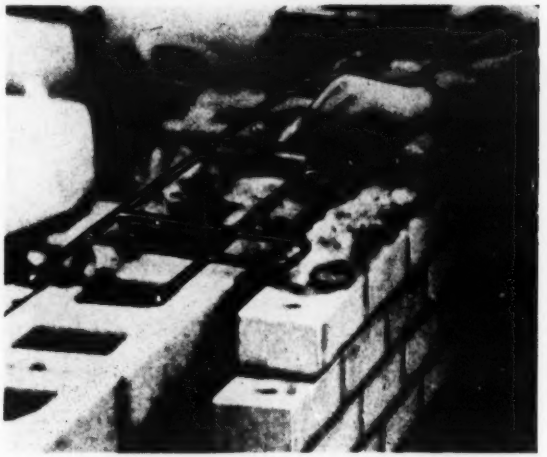

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Lightweight permanent cylinder mold

Forney announces a new lightweight concrete cylinder mold. The weight of the com-

plete mold is 13 lb as compared to 32 lb for the conventional iron mold of the same design.

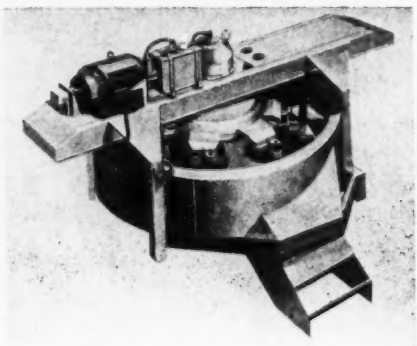
Of three piece construction, the baseplate is iron, and the two halves comprising the barrel are aluminum alloy, which extended actual use shows, is not objectionable from any viewpoint according to manufacturer.

All inside surfaces are accurately machined, and the wall thickness is such that remachining is possible without a dimensional increase beyond specification limits. The swing bolts have a cam action which forces the two halves apart for easy specimen removal.—Forney's Inc., Tester Division, Box 310, New Castle, Pa.

Rex rotary mixer

The addition of a 4-yd Rex Rotary Mixer to its existing lines of equipment for the ready-mixed concrete industry has been announced by Chain Belt Co.

It is said that the 4-yd size of the new rotary will allow double-batching of truck mixers thereby cutting mixing cycles 33⅓ percent over slower triple-batching required by smaller rotary mixers.



Among the features of the mixer are: a slant-slide bottom door, non-leaking and self-cleaning, providing controlled flow with rapid discharge over the full width of the mixer; fast mixing cycle due to double-decked, plow-shaped blades, giving fast mixing action at slower rpm; ease of maintenance with exposed power and drive components; and the lowering of pan which affords unrestricted accessibility to blades and all wearing parts.—Chain Belt Co., Milwaukee 1, Wis.

Set-retarding agent for exposed aggregate concrete

Liquid Edick Surface Retarder can be applied with brush, spray, or roller to steel, wood or masonite forms, or freshly placed concrete surfaces. It is said that set is delayed, but not killed, to a depth of ⅛ in. for about 24 hr. Retarded cement is then washed off with a hose to expose aggregates. Manufacturer claims it will not bleach or discolor cement or aggregates.—Edick Laboratories, Inc., 2358 S. Burrell St., Milwaukee, Wis.

Concrete placing gun

Named the Big Shot, this gun is the latest announced by Air Placement Equipment Co.

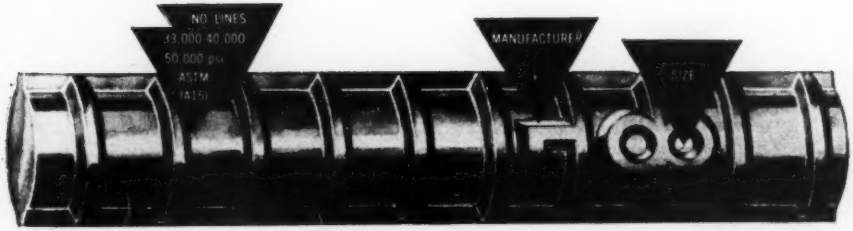
According to the manufacturer, the Big Shot gets and maintains large production from the following features: faster cycling with a 15 in. spherical slide valve; 5 cu ft hopper capacity; and positive feed control. An automatic in-line fog oiler and new valve action result in smoother operation with minimum maintenance. The large "manhole" type access doors to both upper and lower chambers cut clean-up time.

The gun is powered by a 4-hp rotary vane air motor with heavy duty gear box drive.—Air Placement Equipment Co., 1000 W. 25th St., Kansas City 8, Mo.

Bulk truck unloads directly into top of storage silos

A pneumatically equipped bulk cement truck which unloads through a pipeline directly into the top of customers' cement storage silos has been introduced in Springfield, Ore., by Calaveras Cement Co. Using this truck, cement can be blown into customers' silos without the help of conventional screw and elevator unloading systems.

The Fruehauf equipment is completely self-contained, carrying its own compressor for air supply. Calaveras has two additional units for California and Oregon use.—Calaveras Cement Co., a division of The Flintkote Co., 315 Montgomery St., San Francisco 4, Calif.



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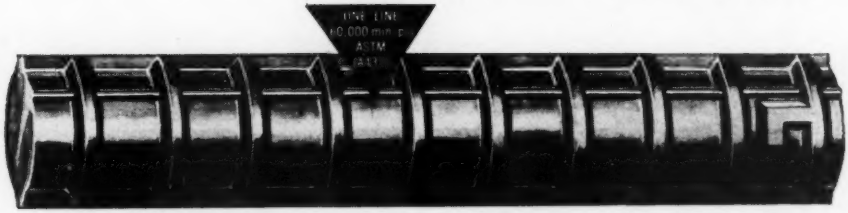

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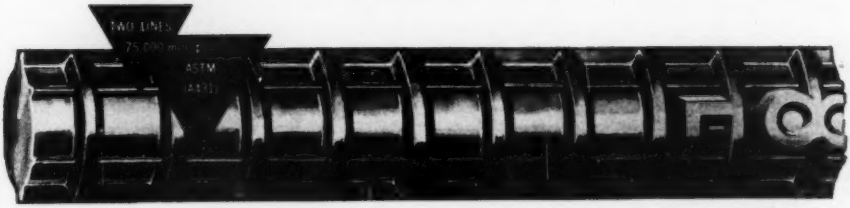

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on remote jobs for Johnson, Drake & Piper, Inc., Oakland, Calif.

In construction of the $8 million House of Correction prison, Susanville, Calif., 21,000 cu yd of concrete is produced at considerable savings by the contractor's portable plant. Batched to rigid State specifications, the 250 cu yd per day requirements are well below the 125 cu yd per hr output capacity of the plant. The fully automatic transit-mix plant with 3½ cu yd weigh batchers has storage for 35 tons of scoop-loaded aggregate and 2500 cu ft of cement. Known as the Noble-Mobile, the plant is manufactured by the Noble Co., Oakland, Calif.—Johnson, Drake & Piper, Inc., Oakland, Calif.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

HOW TO PREPARE CALCIUM CHLORIDE SOLUTION—Prepared especially for ready-mixed concrete producers, bulletin contains illustrated recommendations for procedure in making solution, and a chart on recommended gallons of solution per batch. It includes information on commercial and shop-made automatic dispensers.—Calcium Chloride Institute, 909 Ring Building, Washington 6, D. C.

VERMICULITE ROOF INSULATION DATA (AIA File 37-B-2)—Three technical sheets on vermiculite concrete roof insulation. Two sheets give data and a short form specification for the 1:6 and the 1:8 mix of vermiculite concrete over prestressed concrete double tee or channel roof slabs. The third sheet presents load tables for galvanized corrugated steel roof units.—Vermiculite Institute, 208 South LaSalle St., Chicago 4, Ill.

A SUMMARY OF ENGINEERING RESEARCH 1960—This 144 page book summarizes briefly research projects in the college of engineering at the University of Illinois during the fiscal year, July 1, 1959, through June 30, 1960. This third in an annual series is a general guide to the total research program and not a detailed technical presentation. Each listing of a research project gives the project title, the names of the investigators, a brief statement of the work being done, and a list of publications and theses that have resulted from the research during the year covered by the summary. Copies of the 1959 and 1960 summaries are available. — Engineering Publications, 114 Civil Engineering Hall, University of Illinois, Urbana, Ill.

1961 LIST OF ASTM PUBLICATIONS—The American Society for Testing and Materials has released a 62-page list describing the symposiums, manuals, special publications, indexes, compilation of standards, charts, reference photographs and reports published by the society through the years. More than 300 items are fully described, 40 of which are new and not previously listed. The publications cover all phases of materials and their evaluations and are arranged conveniently by titles and subject—The American Society for Testing and Materials, 1916 Race Street, Philadelphia 3, Pa.

BRICK AND CONCRETE MASONRY (F17.2)—Circular issued by the Small Homes Council-Building Research Council presents basic information about brick and concrete masonry. It tells about kinds of walls—solid, cavity, and veneer; construction essentials, insulation, brick and concrete block sizes and types, joints, patterns, and other matters of interest to home builders and owners. Single copies of the 8-page illustrated circular are 15 cents per copy.—Small Homes Council-Building Research Council, Mumford House, University of Illinois, Urbana, Ill.

SAVE CONSTRUCTION DOLLARS WITH AMERICA'S BASIC CONSTRUCTION METHOD, THE SINGLE CONTRACT SYSTEM—Brochure cites the single contract method as a time and money saver through efficient coordination of the myriad or complex details in a modern construction job by the general contractor. Contract awarding authorities from private industry and from all levels of government are quoted as supporting the single contract method of construction—Associated General Contractors, 1957 E Street, N. W., Washington 6, D. C.

ECONOMICS OF MASONRY INSULATION—The economic details of insulating brick cavity or concrete block walls with a water-repellent masonry fill are discussed in a new Zonolite brochure. Installation costs, savings

in annual heating and air conditioning bills for various parts of the country, and return on investment are covered in the 4-page folder. Tables provide information on the thermal efficiency and heat transmission of various types of cavity and concrete block walls.—Zonolite Co., Dept. MF-45, 135 South LaSalle St., Chicago 3, Ill.

INTEGRALLY COLORED CONCRETE FLOOR CATALOG—Colorundum and its companion products, Colorundum Sealer and Concentrated Colorglaze Wax, are fully described in a 4-page catalog covering the subject of integrally colored concrete floors that require color, hardness, abrasion resistance and ease of maintenance.—A. C. Horn Companies, Division Sun Chemical Corp., 2133 85th St., North Bergen, N. J.

REX CONCRETE BRIDGE-FINISHING MACHINE BULLETIN (61-159)—Four-page bulletin describes the working versatility of the Rex Bridge Finisher and contains detailed specifications and a listing of optional equipment. Manufactured with a built-in width flexibility to take care of most width adjustments, the unit can be easily converted into a standard highway finisher.—Chain Belt Co., Milwaukee 1, Wis.

STEEL FORMS FOR STREET PAVING (No. UF-100)—A completely standardized, interchangeable system of steel forms for street paving is described and illustrated in a 12-page Blaw-Knox booklet. The brochure highlights curb and gutter, flexible radius, fixed radius, straight curb, battered curb and sidewalk forms, as well as details and accessories. —Sales Promotion Dept., Blaw-Knox Co., Construction Equipment Division, Mattoon, Ill.

CALCIUM CHLORIDE FOR CONCRETE CONSTRUCTION—New brief prepared for ready-mixed concrete operators, contractors, and engineers who wish to accelerate the set of concrete, especially when outside temperatures fall below 70 F. Lists major beneficial effects of using calcium chloride for this purpose; contains charts on early compressive strength, calcium chloride with air-entrained concrete, and tells how to prepare and use calcium chloride in standard solution. —Calcium Chloride Institute, 909 Ring Building, Washington 6, D. C.

CLEAR SEAL CATALOG—Complete description of Clear Seal, a transparent, quick drying, synthetic liquid, which cures, seals, hardens, and dusproofs interior or exterior concrete floors or slabs in one operations, is presented in 6-page catalog. Cites advantages, uses, method of application, properties, resistance, packaging, weight, and detailed specification.—A. C. Horn Companies, division Sun Chemical Corp., 2133 85 St., North Bergen, N. J.

■ PRÁCTICAS RECOMENDADAS PARA SELECCIONAR LAS PROPORCIONES DE LOS INGREDIENTES DE LA MEZCLA (ACI 613-54)

Este librete, norma del ACI, trata de las proporciones de los ingredientes incluyendo mezclas conteniendo aire incluido. Contiene tablas, las cuales, junto con los resultados de pruebas hechas en laboratorios con los agregados finos y gruesos, dan información rápida para la obtención de las proporciones. También incluye un método simple para dar las proporciones de la mezcla para trabajos pequeños.

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■ PINTURAS PARA EL HORMIGON

Este Manual es una traducción autorizada de dos publicaciones del ACI: "Guia para pintar el hormigón" y "Recomendaciones prácticas para la aplicación de pinturas de cemento Portland a superficies de hormigón" (ACI 616-49)

La Guía describe los tipos de pintura que no sean a base de cemento Portland que comúnmente se aplican al hormigón. En ella, también se analizan los procedimientos de preparación de superficies, la selección y aplicacion de la pintura así como el repintado. Se trata en forma sucinta su impermeabilidad al agua y si son a prueba de condensación.

El otro librete, una norma del ACI, establece recomendaciones prácticas para el uso apropiado de la pintura de cemento Portland, edad del hormigón, preparación de la superficie, manipulación, aplicación y curado de las mismas. Tres apéndices incluyen comentarios sobre la composición, manufactura y almacenamiento, características generales y factores que afectan la durabilidad.

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